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### FINAL COMPLETION OF THE BUFFALO, N. Y., BREAKWATER EXTENSION TO STONY POINT.*

That portion of the Buffalo, N. Y., breakwater known as the extension to Stony Point was finally completed on Sept. 24, 1903. At 11 a. m. on that date the last stone in the rubble mound or stone breakwater section was laid in the presence of Col. Theo. A. Bingham, Corps of Engineers, U. S. A., the engineer officer now in charge of the district, who has lately succeeded Col. Thos. W. Symons; Assistant Engineers Messrs. J. C. Quintus and Emile Low, Ms. Am. Soc. C. E.; L. L. Davis, Chief Inspector, and Superintendents A. C. Eldridge and Patrick O'Brien of the firm of Hughes Bros. & Bangs, the contractors who have built the work.

As the Buffalo breakwater lays claim to being the longest breakwater system in the world, some few facts relative to this harbor work may be given here. The system consists of five separate breakwaters, as follows:

| | Lengths, feet. |
|---|---|
| North breakwater, timber cribs and concrete superstructure | 2,203 |
| Old breakwater, timber cribs and concrete superstructure | 7,607 |
| Stone breakwater, with gravel heading | 7,250 |
| South Harbor breakwater, timber cribs and concrete superstructure | 2,739 |
| Stony Point breakwater, timber cribs and superstructure | 2,803 |
| Length = 4.28 miles = | 22,604 |

The original construction dates from June 7, 1869, at which date the first crib of the old breakwater was placed. This breakwater was completed on October 13, 1893, nearly a quarter of a century being employed in its construction.

That part of the breakwater known as the extension to Stony Point was designed by Col. T. W. Symons, and the contract for the same was let late in January, 1897, to the firm of Hughes Bros. & Bangs, of Syracuse, N. Y., their bid being $1,765,450.63.

Work on the extension was begun on May 19, 1897, seven working seasons being required for the construction. The cost of the extension in round numbers is given below:

| | |
|---|---|
| Sand catch pier | $6,600 |
| Stone breakwater | 965,000 |
| South Harbor breakwater | 482,000 |
| Stony Point breakwater | 274,000 |
| Total | $1,727,600 |

To this must be added $195,000, the cost of the concrete superstructure on a part of the South Harbor breakwater, which replaced the timber superstructure damaged by a severe gale in 1900. This latter work was done by the Buffalo Dredging Co., of Buffalo, N. Y.

*Several articles describing this breakwater system have appeared in the columns of Engineering News; the interested reader is referred to the following: "The New Stone Breakwater at Buffalo, N. Y.," by Emile Low, U. S. Asst. Engr., in the May 16, 1901, number; "The Concrete Breakwater Construction at Buffalo, N. Y.," by Major T. W. Symons, Corps of Engineers, U. S. A., in the May 29, 1902, number; "The Materials for the Concrete of the Buffalo Breakwater," by Emile Low, U. S. Asst. Engr., in the September 11, 1902, number; "The Accident to the Buffalo Breakwater," in the March 19, 1903, number.

### TWO SYSTEMS OF USING ACETYLENE GAS FOR CAR LIGHTING

At the annual meeting of the International Acetylene Association in Chicago, papers were presented on the use of acetylene gas for lighting railway cars, and an abstract of one of these papers, by Mr. E. G. Fisher, is given below. In another paper, on "Difficulties Encountered in Car Lighting by Acetylene Gas," by Mr. McDonald Elliott, of Santa Rosa, Cal., there were outlined a number of the practical requirements to be met by a successful car lighting system. These are claimed to be met by a system designed by Mr. Elliott, in conjunction with Mr. G. L. Wilson, of Chicago, but this has not yet been put in actual service. Some of the more important features of the Elliott & Wilson system are summarized as follows:

1. There is no calcium carbide in the car, and all recharging with water and carbide is done from the outside, the recharging being done by unskilled labor.

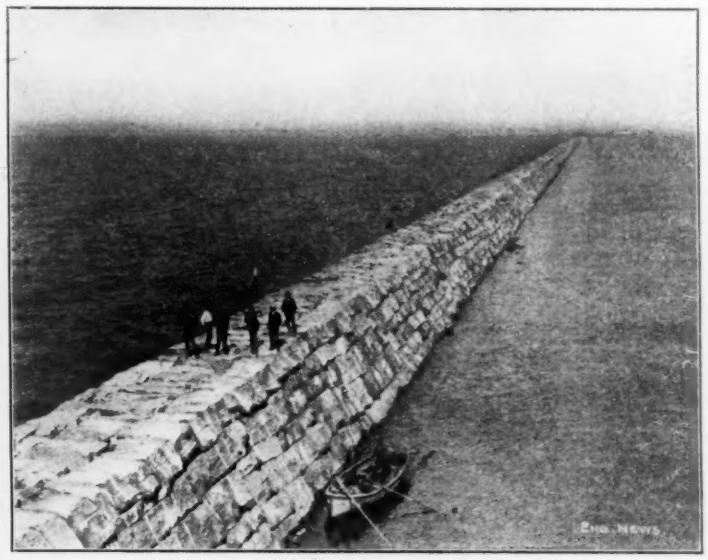
2. The operation is automatic, gas being generated only when required.

3. The plant will run 30 days on one charge of water and 250 lbs. of carbide, this large charge reducing the attendance required. The floor space occupied is very small.

4. The pressure never exceeds 0.30-in. of water, as high pressures destroy burners and waste gas, while regulating devices are liable to fail.

5. The apparatus is all under lock, so that it

Junction of South Harbor Breakwater and Stone Breakwater.



Completed Stone Breakwater.

THE NEW STONE BREAKWATER AT BUFFALO, N. Y. (EXTENSION TO STONY POINT) COMPLETED SEPT. 24, 1903.
Major Thos. W. Symons and Col. Theo. A. Bingham, Corps of Engineers, U. S. A., Engineers;
Messrs. J. C. Quintus and Emile Low, Members Am. Soc. C.E., U. S. Assistant Engineers; Hughes Brothers & Bangs, Contractors.

cannot be meddled with by employees or passengers.

6. The train crew is not required to do more than light and extinguish the lamps.

7. In case the car is overturned the apparatus becomes inoperative, but it will not be affected by minor accidents.

## Compressed Acetylene for Lighting Railway Cars and Stations.

By E. G. Fisher.*

Ever since the discovery of acetylene, it has been a problem to find some safe and economical means of rendering it portable. An endeavor was first made to compress it at high pressure, or even to liquify it, but this had to be abandoned on account of the danger from explosion. In 1896, Messrs. Claude and Hess, two French engineers, conceived the idea of utilizing some liquid as a solvent of acetylene, and after many experiments discovered that acetone (a limpid mobile liquid, with an agreeable odor and burning taste, produced by the destructive distillation of acetates, and procured on a large scale from the aqueous liquid obtained in the dry distillation of wood) has the property of absorbing many times its volume of acetylene gas. This property varies greatly with the pressure and temperature under which the acetylene is applied; the acetone will again release the gas absorbed under pressure, when the pressure is removed. Acetylene dissolved in acetone is inexplosive, as has been shown by Messrs. Berthelot and Vieille, French chemists. It is an inert liquid, so that if an explosion be intentionally produced in the compressed gas outside or above the solution, the solution would not be affected thereby. Thus, if a red-hot wire were inserted into a tank containing acetylene dissolved in acetone, under a pressure of 240 lbs. per sq. in., the small quantity of gas that exists above the liquid would explode, but the liquid would not be influenced in the least. This was a long stride in the direction of safely transporting acetylene, but absolute safety is demanded in a gas-lighting system, especially for transportation, and it was Edward Fouche, another Frenchman, who conceived the idea of packing the gas cylinders with an absorbent or porous material, thereby filling the entire air space, except, of course, the small cells in the porous brick or asbestos; then introducing sufficient acetone into the tank to moisten the cell walls of the porous brick or asbestos. This has been proved to make an absolutely safe structure for storing and transporting acetylene in small or large quantities.

In our work of lighting cars and yachts, we have our cylinders packed with asbestos disks, filling them completely before the heads of the cylinders are sealed. When ready to charge with gas, we first saturate the asbestos with about 40% of the tank's cubic capacity with acetone. One of the small tanks, 6 × 30 ins., equipped for actual service, has a capacity of a little less than ½ cu. ft. It will hold 3¾ gallons of acetone, 40% of which is about 1½ gallons. This cylinder now contains 78 cu. ft. of acetylene gas, under a normal pressure of 240 lbs., or sufficient to supply two 0.5-ft. burners for 78 hrs., continuous burning.

The tanks used by the Pintsch Gas Co. in lighting cars have a capacity of 22 cu. ft., and at 10 atmospheres will hold 220 cu. ft., of Pintsch gas. With the same size tank, and under the same pressure, with the use of acetone and asbestos, on the Commercial Acetylene Co.'s system, 2,200 cu. ft. of acetylene gas can be stored. We go further, and charge the tank at 240 lbs. pressure or 16 atmospheres, which enables us to put 3,500 cu. ft. of acetylene into 22 cu. ft. of space, that has already been filled with asbestos and the asbestos saturated with acetone.

We have equipped a 70-ft. chair car on the Atchison, Topeka & Santa Fe Ry., and on Oct. 25, 1902, it started on its initial trip to Denver. In this car we used eight 2-flame lamps in the body of the car, besides six other lights in the vestibule and closets, giving more than double the candle power furnished by 32 burners of Pintsch gas in the body of the same size car. This car, up to May 25, 1903, had been in continuous service for seven months, traveling between 80,000 and 90,000 miles, making 140 all-night runs, during a season of the longest nights of the year, with but three charges of gas, as follows: Oct. 25, Jan. 5, and March 10. Each charge of gas was about 3,000 cu. ft., averaging 70 days' time at a total cost of $65 for gas for the seven months—less than half the cost of Pintsch gas, and furnishing more than double the candle-power.

PLACING THE LAST STONE OF THE BUFFALO BREAKWATER.

One of our tanks 20 ins. × 10 ft., holding 3,500 cu. ft. of acetylene, contains as many cubic feet of gas as 16 Pintsch tanks, but as we consume only 1 ft. of acetylene to 3 ft. of Pintsch gas, the burning hours of 3,500 cu. ft. of acetylene are equal to 10,500 cu. ft. of Pintsch gas. In other words, one tank under our system is equal in gas storage to 50 tanks of same size filled with Pintsch gas. According to recent tests made by Mr. J. W. Morehead, gas and electric engineer, of Chicago, and Mr. F. W. Thomas, Engineer of Tests of the A., T. & S. F. Ry., the relative candle-power of the two gases is nearly 1 to 7, 8.6 c. p. per cu. ft. of Pintsch gas, as against 56 c. p. of acetylene. Therefore, we find that 3,500 cu. ft. of acetylene gives a total of 196,000 c. p., as against 90,300 c. p. for 10,500 cu. ft. of Pintsch gas. The Pintsch gas is sold to railway companies at $5.09 per 1,000 cu. ft., which would be $53.44 for the 10,500 cu. ft. Acetylene gas can be manufactured at $8.00 per 1,000, giving $28 for the 3,500 cu. ft.

About 1% of the quantity of acetone used passes out with the gas. As 1 lb. of acetone will produce a larger volume of vapor than 6 lbs. of carbide and the acetone vapor must mix with the gas to be carried away, and as the burners will only consume a certain quantity per hour, we gain an increased burner-hour for a given amount of acetylene. Furthermore, acetone (being of the alcohol family) prevents the carbonizing of the acetylene at the burners.

The generating and compressing plant necessary for the operation of this system is very simple and inexpensive. Any good acetylene generator and gas compressor of proper capacity can be utilized. The entire plant (exclusive of building) suitable for lighting shops and charging tanks for cars, and for transporting over a railway system, can be furnished complete from $1,500 to $8,000, according to capacity. It requires but two men to operate such a plant.

STATION LIGHTING.—This system is very economical for lighting railway stations, shops, signals lights, towers, etc. The generating plant being located near the shops, the shops can be lighted in the ordinary way, using the ordinary gas holder, and the tanks can be charged and transported along the line to stations and other buildings for lighting. In this way, a railway company can manufacture acetylene for use in every department, thereby gaining the best light known, at less than half what they pay for gas and electric lighting companies in the large towns along the road.

At Morris, Ill., we equipped the railway station for the Chicago, Rock Island & Pacific Ry. with our system. The station having been piped for city gas, it was only necessary to place one of our 20-in., 10-ft. tanks, with regulating valves and pressure gage attached, in one end of the baggage room, connect it with the station piping, and change the burners on the fixtures. This station has 20 lights, and the average monthly gas bill for city gas furnished by the city gas company has been for the past year, $26.50. Our tank holding about 3,000 cu. ft. of gas and costing about $25 for the gas, was placed in this station on April 28 last, furnishing a 50% better light for three months (up to July 28), thereby making a saving of 66%. In other words we furnished acetylene light for three months for less than they have paid the local gas company for one month for a poorer light.

YACHT AND OTHER MARINE LIGHTING.—We have equipped many boats, of different kinds and sizes; some of them were equipped with electric lighting plants, but have since discarded the electric plant. The great feature in vessel lighting by this system, as in car lighting, is its simplicity and safety and economy; it has no working parts to get out of order, and can store a large quantity of gas in small space.

This system can also be used on automobiles, carriages, and anywhere a good light is required, either stationary or portable.

In conclusion, the following points may be presented regarding the safety of this storage system:

1. It has been approved and licensed for city use by the City of Paris, after extensive tests by Mr. Vieille, the engineer having charge of explosives. 2. It has been approved by the Government of Great Britain, and the Secretary of State has officially found that our tanks are safe. 3. It has been approved by the Board of Engineers of the Underwriters' Laboratories at Chicago, acting under the National Board of Fire Underwriters, which has directed that rules be prepared for this system on cars and ships. 4. It has been approved by the United States Lloyds; by La Paternelle, one of the largest insurance companies of Paris; by the Municipal Explosives Commission of the City of New York, and by the U. S. Treasury Department.

The committee on Science and Arts of the Franklin Institute has awarded the inventor the John Scott Legacy Medal.

*Commercial Acetylene Co., Chicago.

STREAM TIN WAS DISCOVERED IN ALASKA last year, and now it is reported that great ledges of tin ore have been found at Cape York on Bering Sea. Numerous individual placer miners are reported to have made small fortunes during the past summer; two men, for example, having taken 22 tons of stream tin from claims along one of the creeks in the Cape York region, using the crudest hand methods. Hydraulic machinery will be taken into the district next season, when the extent and value of the tin deposits will be ascertained.

## INTAKE TUNNEL FOR THE CHAMPION MILL, AT PAINESDALE, MICH.*

By F. W. O'Neil, Jun. Am. Soc. M. E.†

The two things of primary importance to a stamp mill in the Lake Superior district are the unlimited supply of water and of room for the waste sands. Each stamp-head treating approximately 500 tons of rock per day will make, at 20 cu. ft. to the ton, 10,000 cu. ft. of sand per day, or 3,000,000 cu. ft. per year. The amount of water required varies considerably, according to the practice at various mills, but up to a short time ago the quantity required was approximately 5,000,000 gallons for each stamp per day of 24 hours. Hence, in determining the site for the Champion mill in the spring of 1901, it was taken for granted that it should be located on the shore of Lake Superior. No location on Portage Lake could be secured where the waste sands would not soon encroach on the harbor lines. Hence the mill was located on the shore of Lake Superior in Section 25, Range 55 N., 36 W., since the topography and geology at this place made it most desirable for a stamp-mill site. The country rock is of sandstone dipping westward into the lake at an angle of 4°, the strata are from a few inches to several feet thick with occasional soft seams. The

*Abstract of a paper prepared for the annual meeting of the Lake Superior Mining Institute, August, 1903.

†Chief Engineer, Champion Copper Co., Painesdale, Mich. Mich.

shore of the lake is a sandstone bluff from 12 to 30 ft. high, and the nearest harbor is at the Portage Lake ship canal, 11½ miles away. The survey showed the lake bottom to be of sandstone sloping quite uniformly, so that at 1,000 ft. from the shore the depth of the water is about 22 ft.

The problem of how to get the water out of Lake Superior into the mill was a new and rather difficult one. The Atlantic and Baltic mills, at the time the only mills operating on Lake Superior, were taking their water from the Salmon Trout River by gravity from the steel dam at Redridge.* Owing to the thick, heavy ice cake that forms along the shore of Lake Superior in the winter, a simple trench extending from the pumping plant to the lake shore similar to that employed in some of the mills on Portage Lake was out of the question. Hence the problem resolved itself into the necessity of installing some device for an intake which would tap the lake at a sufficient distance from the shore to be out of danger from ice and sand. It was thought, also, that the intake should be so designed as to make it easy to remove sand from it should conditions in the future demand it.

Three general methods suggested themselves, of which the first was an intake pipe. This would have to be laid in a trench excavated in the lake bottom far enough out into the lake to get into 12 ft. or more of water, so as to be out of danger from crushing by ice. The remainder of the pipe to be laid in the lake bottom and fastened thereto by means of chains and concrete piers, and finally at its outer end enter an intake crib of either the submerged or exposed type fastened to the lake bottom. This type of construction was afterwards adopted at the Tri-Mountain Mill. The second method was a combined tunnel and pipe. In this case the tunnel would take the place of the trench above mentioned, being driven under the lake bottom and piercing it in sufficient depth of water to protect the pipe (which would be connected to it) from ice. The pipe would then follow the lake bottom, as before, as far as the crib. This construction was used at the Duluth water-works, where the pipe was 60 ins. diameter, and the crib was of the submerged type in 56 ft. of water.†

The third method was to run a tunnel from a shaft sunk on the shore out under the lake to a submerged crib. After careful consideration and investigation of all the methods, this seemed to be the best and was adopted. The principal argument against the driving of a tunnel was that in the sandstone bottom of the lake there are occasional cracks showing, some in line with the course of the tunnel and others intersecting it at an angle of about 45°. These fissures vary in width up to 3 ins. It was predicted that if the tunnel should intercept one of these fissures, it would be necessary to put in an air lock and complete the work under pneumatic pressure. This method has been used in the construction of a good many tunnels and has nearly always resulted in loss of life, due either to accident, or as the result of a peculiar disease which sometimes attacks men who have worked under air pressure.

In constructing the pump house it was necessary to excavate into the country rock to a depth of from 10 to 12 ft. over an area of 50 × 75 ft. When the earth was stripped away from the surface of the rock, the rock surface was seen to be more or less fissured the same as in the lake bottom. On examination it was found, however, that these fissures never attained a depth of over 4 ft., as they were never continuous through more than one stratum in the sandstone. The principal advantages of the tunnel therefore seemed to be the following: (1) The cost: Provided the tunnel could be driven without the use of air locks, our estimates showed that it would be much cheaper to drive a tunnel 5½ × 6½ ft. rather than to lay a 40-in. pipe. (2) The necessarily large size of a tunnel as compared to a pipe. Our tunnel is 5½ × 6½ ft., and therefore has the same cross-sectional area as an 81-in. pipe, but such a pipe is entirely too large to lay successfully. Hence the tunnel would fulfil our needs for practically all time, whereas a 40-in. pipe might have to be duplicated in a few years. (3) A tunnel inclined downwards toward the shore end in the direction of flow of the water in it, having to be driven in this way so as to handle the water during construction. Hence should sand get into the tunnel, it would tend to move along with the water, making it easier to adopt means of depositing the sand in a suitable sump at the shore end, from which it could easily be removed with a sand pump.

The possibility of sand entering the intake was fully considered. The waste sand from four stamps would amount to about 12,000,000 cu. ft. a year. If we assume the sand to be distributed a mile along the coast, it would take a trifle over four years for the lake to fill with sand to a distance of 1,000 ft. from the shore, the sand having an average depth of 10 ft. Investigation showed, however, that such figures mean very little on account of the tremendous action of waves in distributing sand laterally along the shore. At the Atlantic Mill the sands have been distributed by means of wave action several miles. Here also one storm has been known to tear away 75 ft. of the shore line. The Atlantic Mill is in a bay facing north, while the Champion Mill is on the exposed shore line facing west. As the prevailing storms come from the west or northwest, it would be expected that wave action on the sands would be much more violent in our case. It is to be regretted that no concise data are obtainable at this time on this interesting subject.

For the foregoing reasons it was determined to sink a shaft communicating with the sump in the pump house, and from it drive a tunnel under the lake bottom for 1,020 ft., then to make connections with the lake bottom by means of a submerged crib. The principal reasons for using a submerged crib were: (1) Its lower cost as compared to an exposed crib; (2) To avoid the danger from battering by waves and ice to which any exposed intake crib would be subjected. There will probably be less danger also from needle and anchor ice. It is not the purpose of this paper to enter into a discussion of frazil, or needle ice, and anchor ice, as much has been written on this subject. But since the two are often confused, a statement of the general properties of each may not be out of place. Frazil, or needle ice, consists of small needles, or flakes, of ice formed when the water is below the freezing point, but in too great commotion, due to wave action or currents, to allow the needles or crystals to form solid ice. It is never formed under solid ice. Comparatively weak currents attract it, and once drawn to an intake grating, it clogs up the openings by forming more or less solid ice. This clogging effect is greater the smaller the mesh of the intake screen and the nearer it is to the surface. True anchor ice is ice formed on submerged bodies due to the radiation of heat from them through the water into the air. Unlike needle ice, it may be formed under surface ice when the surface ice is thin and clear. It is not formed when the sun is shining. It forms most readily on dark metallic surfaces, and never forms in conduits of brick or stone, another advantage of a tunnel over a pipe. Neither needle or anchor ice form at a depth of 40 ft. or more. Trouble with these forms of ice, particularly the needle variety, has been the cause of the unsuccessful operation of many intakes on the Great Lakes.

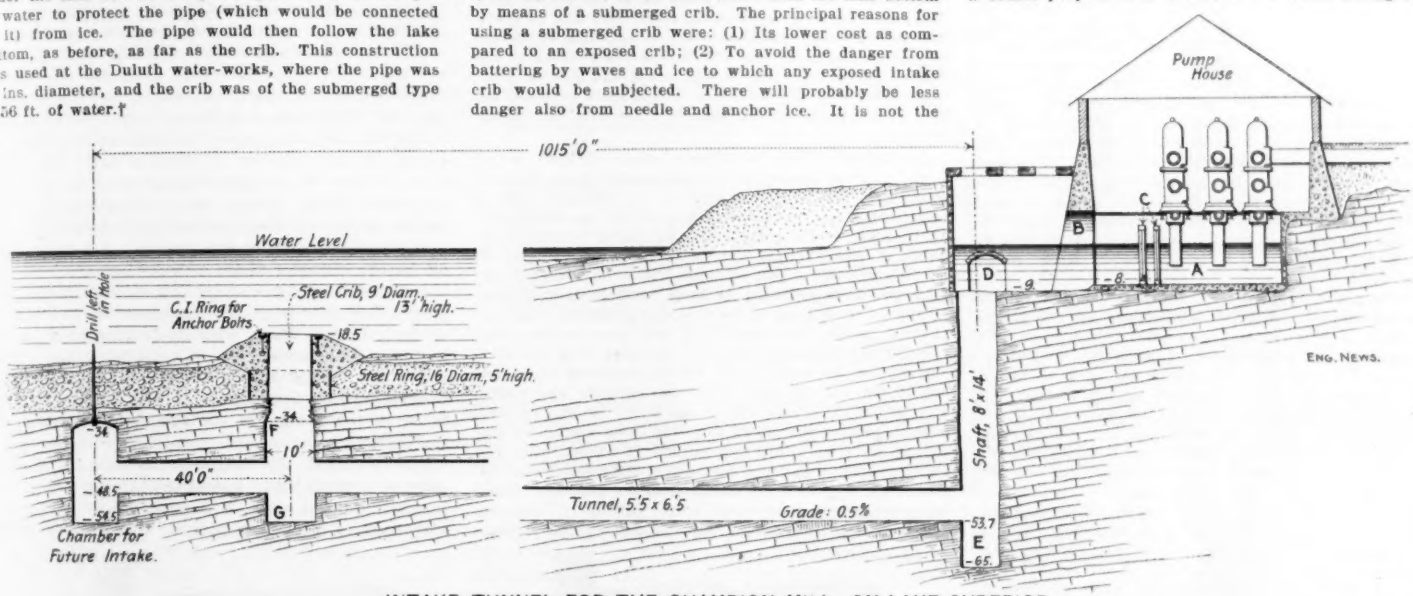
Investigations have shown, however, that when the velocity of the water entering intake ports does not exceed 0.5 ft. per second, no trouble is to be feared from this source. Hence the intake crib was so designed that this velocity was not exceeded. A coarse mesh screen was used so that the needle ice could not easily form across the openings, but allow its passage through to the tunnel. Once in the tunnel it would probably melt before reaching the pump.

The work was started in November, 1901, and completed November, 1902. The tunnel proper was started the middle of January and completed the following August, the rest of the time being used by the sinking of a shaft and the construction of the intake.

The shaft on the shore from which the tunnel is driven is connected with the sump, or pump well, A, in the pump house by a channel passing beneath an arched opening, B, in the pump-house wall. This sump is 13 ft. 9 ins. wide and 8 ft. deep below the water level, the sides and bottom concreted to a depth of 12 to 15 ins. Two screens, C C, serve to keep out foreign substances and fish from the pump. These gates are movable, sliding on cast-iron guides secured to the side of the sump. These guides are so constructed that one of the screens can be removed and a wooden bulkhead inserted in its place so that the sump may be cleaned out. The shaft is 8 × 14 ft., and is 65 ft. deep below the water level, and 80 ft. beneath the collar. In passing it should be mentioned that the tunnel, D, was driven below the water level a distance of 28 ft. at right angles to the sump. This was done so that if it became necessary in the future to build another pump house, due to increased milling operations, connection could be made with no trouble to the sump. To insure this, the mouth of the tunnel, D, is so arranged that a wooden gate, or bulkhead, can be placed over it to exclude the water.

In sinking, a South African or Kimberly skip was used, having a capacity of about two tons. Many seams were met between the successive layers of sandstone which carried a good deal of water. Most of this water, however, came from the land side of the shaft. This is readily accounted for by the dip of the strata. This water was, however, all successfully handled by pumps. At the bottom of the shaft the sump, E, was sunk. This served a double purpose of a catchbasin for water during construction, and it is also anticipated that it will catch any sand which may find its way into the tunnel in the future. The entire work of sinking and drifting was done on the company's account until the tunnel had been driven 162 ft., when the men wished to go on contract. This request was granted and a rate of $4.50 per lin. ft. was established. After the tunnel had been driven 600 ft., the contract price was raised to $5 per ft. The miners made from $2.71 to $4.92 per day, according to their efficiency. The rate of drifting per month varied from 116 ft. to 177 ft. There were three shifts of miners, two on each shift. Two trammers were employed until the tunnel had been driven 600 ft., after which four were employed. The trammers received $2.15 a day. Considerable trouble was experienced in obtaining men. This is shown by the fact that although only six miners were employed at a time, yet 74 different miners were employed during the work.

Considerable water was encountered in the tunnel, though not as much as was anticipated, the flow at no time exceeding 200 gallons per minute. Since the tunnel was driven on an upgrade of 0.5%, and the strata dipped downward at an angle of 4°, we regularly broke through to successively higher strata on the hanging. Nearly all of the water came from the joints between the strata. Very little timber was used in the tunnel; occasionally a cedar post. The real difficulties of the construction began to present themselves when the tunnel was completed and the connection to the lake bottom was attempted. At the end of the tunnel there was left 20 ft. of rock between the top of the tunnel and the lake bottom. It was considered safe to make a raise 8 ft. high, thus leaving 12 ft. of rock to be removed by operations from the lake. In order to have a low velocity of approach to the intake




INTAKE TUNNEL FOR THE CHAMPION MILL, ON LAKE SUPERIOR.

*Engineering News, Aug. 15, 1901.
†Eng. News, May 5, 1898.

orifice it was planned that this intake should be 9 ft. in diameter. The raise was therefore made 10 ft. in diameter and the rock trimmed off until it was as round and true as possible. Two of these raises were made and although one intake was sufficient for the contemplated mill, it was thought that our conditions in the future might demand another opening in the tunnel, and hence all underground work had to be done at this time.

After the raise was completed up to F, a hole was drilled in the center of it straight through to the lake. After drilling 3 ft., the drillings showed sand and gravel. This was rather surprising, since the bottom of the lake had shown only clear sandstone. Subsequent developments showed that the lake bottom was composed of a layer of sandstone about 1 ft. in thickness, covered with boulders, underneath which was a bed of young conglomerate about 8 ft. thick. This conglomerate was made up of pebbles, some several inches in diameter bound together with sand and carbonate of lime. It was not very strong, but entirely too compact to be handled by an orange-peel dredge.

After the drill hole was through to the lake bottom, a drill was pushed up through this hole and then the hole plugged with wood. This drill projected several feet above the lake bottom and served as a guide for the diver in locating the raise. We were now ready for operations from the lake. The tunnel was cleaned out thoroughly and tracks taken out and all the timber work, piping, pumps, etc., in the shaft removed and the tunnel allowed to fill with water. A clam-shell dredge was first employed which removed all the boulders from the lake bottom. These boulders vary in size, some of them weighing over two tons. The dredge also removed the foot of sandstone after the same had been loosened by charges of dynamite placed by the diver.

The most difficult part of the operation was sinking down through the conglomerate. As before mentioned, an orange-peel dredge had little or no effect on it, but a few charges of dynamite made it a sand bank, and it looked as though we should have to remove an enormous quantity of it in order to get to the rock surface. Dredging was kept up until an area of 40 ft. in diameter had been removed, having a depth over the raise of 5 ft. A steel ring 10 ft. in diameter by 5 ft. high, made of ¼-in. boiler plate, was then lowered over the drill. This was lowered by the dredge, and before being let down, two struts of wood were placed in it, intersecting at right angles. These served to keep it in shape and also determined the center of the ring so that the diver had no trouble in centering it over the up-raise by means of the projecting drill.

This ring was finally worked down to the rock by the following method. A pump was set up on the dredge and water at high pressure sent down to the diver, who was inside the ring. He loosened the broken conglomerate under the edge of the ring and heaped it up in the center from which it was removed by the dredge. Occasionally the orange-peel bucket was opened wide and dropped through the water, on to the edge of the ring, thus forcing it down. The position of this ring was always shown by four buoys which were attached to cords of equal length fastened to it.

Dredging inside of this 10-ft. ring in 25 ft. of water with more or less sea running was no easy task. Owing to the lateness of the season and the many storms, the dredge and diver could only work about two days a week. It must be remembered that since the nearest harbor was 11½ miles away, over three hours' trip for the dredge outfit, they did not dare to venture out in questionable weather. Finally the ring was resting on the sandstone and all the broken conglomerate removed from inside of it.

Then to remove the 3 ft. or 4 ft. of solid sandstone a charge of eight sticks of dynamite was laid on the rock. Exploding this made no impression on the rock, but burst the steel ring and scattered the plates of which it was composed all over the lake bottom. Nearly all the rivets were sheared off on all the seams. The seams were lap-joint and riveted with a single row of ½-in. rivets, having about 4-in. pitch. The broken conglomerate was next scraped away from over the center of the raise and a charge of 110 sticks of dynamite exploded on the rock.

This did not remove over 6 ins. of the sandstone, and we decided therefore that we should have to adopt a slower method of drilling holes in the rock for the dynamite charges. We next worked down another steel ring as before and started to drill through the sandstone. This drilling was done from a large raft which had on it a gin-pole about 25 ft. high. A 2½-in. pipe was let down on the rock surface and secured to the raft. This pipe served to guide the drill and also keep the few inches of broken conglomerate which lay on the surface of the sandstone from filling up the hole after it was drilled. A 1-in. hole was bored in the side of this pipe about 18 ins. above its lower end, through which the sludge from drilling worked out. The drill was jumped up and down by means of a tackle fastened to the gin-pole on the raft. In this way we could drill from 2 ft. to 3 ft. an hour. We were now much surprised to find only 16 ins. of rock where we expected to find 3 ft. Investigation showed that the reason for this was that the charges of dynamite laid on the lake bottom had broken from the top of the raise.

As this sandstone was so thin a quicker method than drilling was adopted. A piece of 4-in. shafting about 20 ft. long was fastened to the dredge rope in the place of the bucket. This was allowed to drop vertically about 20 ft. through the air and the remaining distance through the water. The second blow broke the sandstone floor and we had very little difficulty in removing all of the sandstone in this way. Finally the edges of the hole were trimmed up by drilling and very light charges of powder. All this rock, of course, fell into the sump, G, sunk to receive it.

We next made a cylindrical wooden stave crib 16 ft. high and 9 ft. in diameter. It was planned to sink this crib in an upright position centrally over the hole in the lake bottom and then to surround it with concrete. The crib contained a ballast floor and was also provided with a tapered king stave. This ballast floor was so arranged as to collapse when pulled out, from the top, and by having the dredge pull out the king stave, the whole would collapse and leave a concrete tube, or crib, connecting the tunnel to the lake bottom. On a fair day, therefore, this crib was towed out and sunk into position by loading rock on the ballast floor. It was then chained into correct position to the steel ring and we started to fill the space between the crib and the ring with concrete. This concrete was made by mixing the gravel dredged from the lake bottom with cement. It was then placed in burlap bags and passed down to the diver on a guided line, the diver distributing and packing it thoroughly around the crib.

In this way about 3 ft. of concrete was packed around the ring, when a heavy sea came up forcing us to suspend operations. It was the diver's opinion, however, that the crib would be secure against any ordinary storm. The following morning a very high sea was running and the beach was covered with pieces of the intake crib. The writer is of the opinion, however, that this crib was not torn from its place by wave action, but by failure of the iron hoops on the crib, due to the swelling of the wood. Instructions had been given to the men to thoroughly wet this crib every day that it stood on shore before it was used, but the work had not been thoroughly done.

A steel crib 9 ft. in diameter, made of ¼-in. boiler plate, securely riveted and reinforced by three rings of angle iron, was at once made. It was planned to leave this steel crib in permanently and it was therefore made 13 ft. high, which would bring the top of it about 4 ft. above the lake bottom and 18 ft. from the surface. It was thought that a 4-ft. elevation above the bottom of the lake would be sufficient to keep boulders from rolling into it. This crib, as in the case of the wooden one, was chained to the outer steel ring and then entirely surrounded with concrete, the concrete extending out over the edge of the outside ring and up the slope of the pit dug in the lake bottom. When the concrete was within 4 ft. of the top of the crib, a cast-iron ring was lowered over it, so that the top of this ring came flush with the top of the crib. This ring carried 12 anchor bolts 1¼ ins. diameter, with large washers at their lower end, so that it could be securely anchored to the concrete. Then the concrete was brought up to the bottom of this ring and thus sloped all around down to the original bottom. In all about 200 yds. of concrete were used around the crib.

The sump, G, was then thoroughly cleaned out by means of the orange-peel dredge. In doing this work the dredge was working in 55 ft. of water and working through a 9-ft. opening, which was rather a difficult operation. Next a grate composed of 1¼-in. square bars 6 ins. apart, held together by bars, ½ × 1½ ins., spaced 12 ins. apart, laid at right angles to the first, was securely fastened to the iron ring. This serves only to keep out sunken logs, etc., any small foreign substances being kept out by means of the screens in the pump-house sump. The detail of the cast-iron ring is such that should conditions in the future demand it, it would serve as a seat for a large wooden valve which could be placed over the intake and the entire tunnel pumped dry.

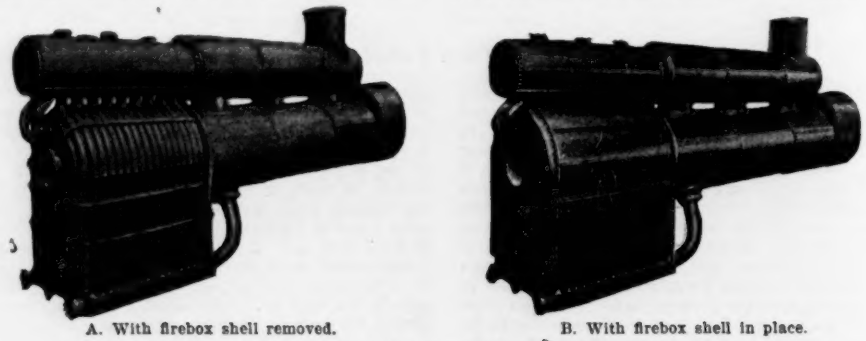
We are now using in our mill, 3,675,000 gallons of water per day for each of the four stamps, or a total of 14,700,000 gallons. The free area through the intake crib grating is 45 sq. ft. Hence the velocity is ½-ft. per second. By putting in the second intake crib enough water can be obtained to supply eight stamps, 29,400,000 gallons, for which the tunnel was designed. In this case the velocity in the tunnel would be 1.3 ft. per second, which is low. It is the writer's opinion that the velocity through the crib grating could be considerably increased without giving rise to any trouble.

The cost of this work will be of interest. The cost of the shaft and tunnel was as follows: Labor, $10,469; supplies, $4,956; total, $15,405. This includes all the work done on the auxiliary tunnel and the connection of the shaft to the pump-house sump. All the work done at the end of the tunnel, after it was completed, in making the sumps, raises, etc., was charged to the intake. This cost was as follows: Labor, $3,769; supplies, $3,294; dredge, $3,920; total, $10,983. The average cost per foot of tunnel was $11.71. The cost of the shaft per foot was $16.50, this including the pumps, piping, timber work, skip, etc. The average daily wage for the tunnel and shaft work was $2.58, while the average daily wage for the work at the crib was $3.23, including the diver, and $2.38 not including the diver. In conclusion we have been using water through this tunnel since last November, and have had no difficulty whatever, the ice last winter giving no trouble, and we have so far spent nothing on its maintenance.

## A LOCOMOTIVE BOILER WITH WATER-TUBE FIREBOX.

A noteworthy innovation in locomotive boilers has been worked out by Mr. J. Brotan, an Austrian engineer, and has been given an extended trial on the state railway lines of Austria with practical success, it is stated. The essential idea of the Brotan design is to replace the ordinary firebox of flat sheets staybolted together, by an aggregation of water tubes forming the furnace walls. It will be admitted, of course, that the locomotive firebox as now built is by far the weakest and most troublesome part of the boiler, and that the replacement of its flat surfaces by water tubes will be of great advantage as far as concerns the surfaces exposed to the fire. The difficulty in any water-tube arrangement is the construction of the details, as is evidenced by the course of development through which the stationary water-tube boiler has passed. A recent article by the inventor of the new locomotive boiler in the "Zeitschrift des Oesterr Ing. u. Arch. Vereins," giving full details of the construction, enables us to describe the boiler with due reference to these important features. We may remark here that while the new design deals primarily with the firebox, certain important alterations are necessitated in the design of the remaining portions of the boiler.

Briefly stated, the Brotan locomotive boiler consists of a cylindrical barrel containing fire tubes through its full area, a firebox whose walls are composed of vertically-placed water tubes bent into the form of an inverted U, and a steam drum extending longitudinally over barrel and firebox. The firebox wall tubes, which we may call box tubes, terminate below in a small longitudinal mud drum on each side of the ash-pit; at the highest point of each tube a connection extends up to the rear portion of the steam drum. The barrel of the boiler communicates with the steam drum by a number of short uptakes. Connection is also made between the bottom of the barrel and the two mud drums. Complete circulation is thus provided for: water from the bottom of the barrel flows down to the two mud drums, thence up through the box tubes where steam is produced: the steam passes up in the box tubes and into the steam drum. Steam generated in the barrel passes directly into the steam drum through the



A. With firebox shell removed. B. With firebox shell in place.

Fig. 1. Views of Brotan Locomotive Boiler.

uptakes. The mean water level is about at the middle of the steam drum, so that the entire barrel and its fire tubes, as well as the box tubes and their nipple connections to the steam drum, are submerged. The firebox proper (composed wholly of water tubes except that a small portion of both the front and the rear wall is of fire-brick) is enclosed by a steel-plate shell which protects the box tubes from the outside air. The shell does not form the wall of the fireplace, however, as the joints between the box tubes are sealed by copper strips. Only the inner half of each box-tube is, therefore, exposed to the direct heat of the fire. The shell extends down to form the ash-pit and also carries the grate, which latter is at a level a little above the mud drums.

In general form the boiler is quite similar to the ordinary locomotive boiler, and it can be fitted to replace any existing boiler without altering the rest of the engine. The external appearance of the boiler is shown by the two views in Fig. 1; the view A shows the boiler with firebox shell removed, while view B shows the complete boiler with this shell in place.

DETAIL CONSTRUCTION.—The barrel of the boiler is cylindrical and of ordinary construction, except that its cross-section is completely filled with fire tubes instead of leaving a vacant space at the top, such as is necessary to form the steam-space in the common form of boiler. The tube plates are of soft steel, ¾-in. thick, and flanged to rivet to the boiler shell. Three uptakes, which are saddle-shaped steel castings, are seated on the shell to carry the steam drum, this being of smaller diameter than the barrel. At the forward end of this drum is the steam dome, from where the steam pipe leads out through the forward head of the drum and into the smokebox. The latter is elliptical in section, i. e., it projects above the boiler shell, so that the pipe connections do not blank any of the fire tubes. The exhaust nozzle is placed in the smokebox in the usual manner. The steam and exhaust pipes are not shown in the drawing of the boiler, Fig. 2, but their arrangement will be evident.

The water tubes forming the walls of the firebox are seamless steel tubes, about 3½ ins. in diameter and about 3-16-in. metal. Each tube is b nt into a U-shaped element, whose bent portion has the same radius as the boiler shell, while the vertical legs extend down the proper distance to connect to the mud drums. The width between the vertical portions of the tubes depends on the desired width of firebox. The simplest arrangement is to make this width equal to the diameter of the boiler shell, but a narrower firebox is easily arranged for; Fig. 3 shows the shape of the individ-

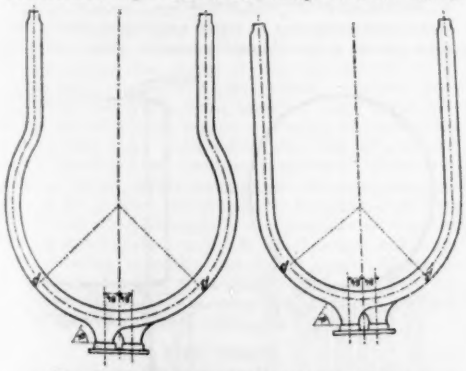


Fig. 3. Two Forms of Firebox Element.

ual tubes or elements in the two cases. At the top of each element is a nipple projecting upward for connecting the element to the steam drum. Since the elements are set close together in assembling to form the firebox, these nipples require to be staggered, alternating to opposite sides of the center line. This is indicated in Fig. 3, which shows the nipples set about 3¼ ins. to one side of the middle point of the element.

The firebox consists simply of a row of such elements, one behind the other; the forward element is placed directly against the rear tube-plate of the barrel, just outside the outer ring of fire tubes, while the rear end of the firebox is closed by a series of successively smaller elements of similar shape, nested one within the other. These latter leave a circular fire-door opening, opposite the center of the barrel, and below the fire-door an open space between the legs of the smallest element. This latter opening is bricked up with fire-brick, leaving a circular fire-hole which is lined with an iron ring for attaching the door. In assembling the firebox, the joint between adjacent tubes is made tight by a copper strip 1-16-in. thick, as shown in Fig. 4; this strip may be calked after the assembled firebox is properly strapped and stayed.

The rear half of the steam drum, that portion of it which extends over the firebox, is somewhat less

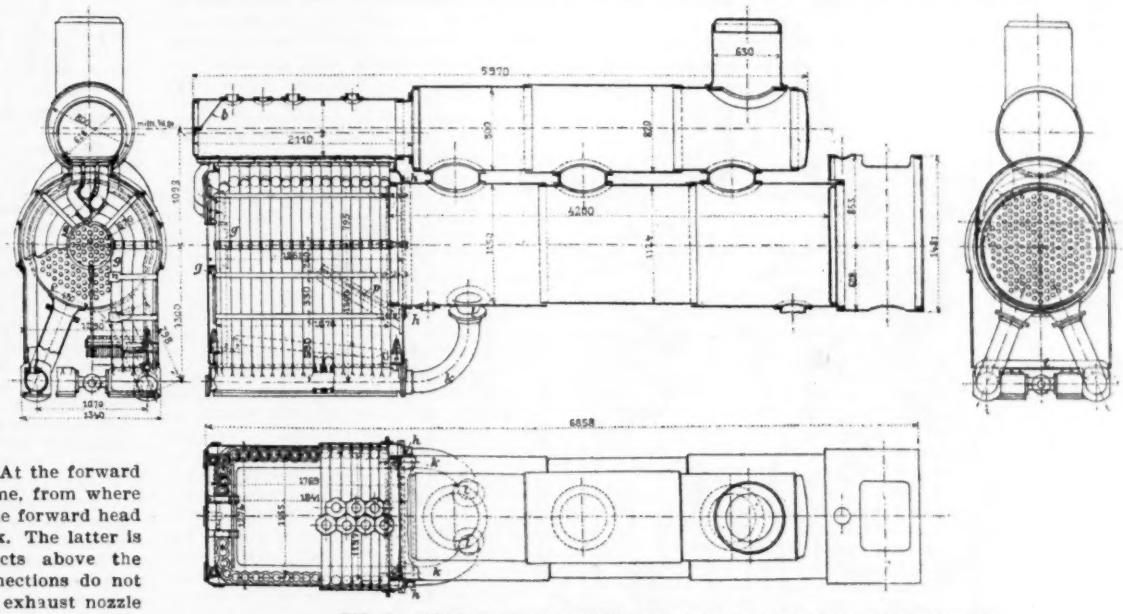


FIG. 2. SECTIONAL DRAWINGS OF BROTAN LOCOMOTIVE BOILER.

in diameter than the forward portion (as will be seen in Fig. 2), in order to allow space below it for the connections of the box tubes. This part of the steam drum, moreover, has a flat bottom, formed of a steel casting, in order to simplify the connections. In the top of the drum are several hand-hole openings which give access to the box-tube connections for cleaning. The water-glass is attached to the rear head of the drum, and a baffle-plate fixed within the drum shields this connection from any splashing which may be caused by the steam rising up from the box tubes.

The mud drum at the bottom of the firebox is of cast steel. It is circular in cross-section, about 8 ins. in diameter, and extends along the bottom of each side wall of the firebox as well as the tube portion of the rear wall. It is preferably made in two L-shaped portions, one on each side of the firebox, joined under the fire-door by a section of pipe fitted with a T-flange for a blow-off connection. Thus the mud drum as a whole is horseshoe shaped in plan; to stiffen it in a lateral direction the two legs of the horseshoe are tied together, at the forward end, by an iron tie-bar. The upper surface of the mud drum is provided with reinforced openings to receive the bottom ends of the box tubes (see Fig. 5), and opposite each of these openings is a hand-hole in the lower side of the drum.

The enclosing shell of the firebox, made of steel plate, follows the outline of the firebox proper, but is everywhere 2 to 3 ins. distant from the box-tubes. At the forward end its front wall, ⅜-in. thick, is fastened to the barrel of the boiler by riveting to an angle-iron ring on the barrel (Fig. 2). The rear wall, also of ⅜-in. plate, is bolted to the fire-door ring. The side and top sections, which may be of thinner (¼ to 5-16-in.) metal, are joined to the rear portion of the steam drum by a short vertical gusset on each side, enclosing the nipples which connect the box tubes to the steam drum. The upper portions of the shell are bolted together in such a way that they may be removed for inspecting the outside of the box tubes. The bottom sections of the side walls of the shell are reinforced for attachment to the locomotive frame. Along the lower edge of the front wall of the shell extends an angle-iron edging; on this is built the fire-brick wall which forms the front of the firebox below the barrel. A similar angle at the lower edge of the rear head of the shell carries the brick portion of the rear wall of the firebox, below the firedoor and between the legs of the innermost tube element. These angles also form the support for the grate bars (see Fig. 2), and give attachment to four hangers, one at each corner of the firebox, by which the mud drum is suspended.

CONNECTIONS.—The connection of the box tubes to the steam drum is made, as already explained, by a flanged nipple at the top of each element bolting to the lower face of the steam drum.

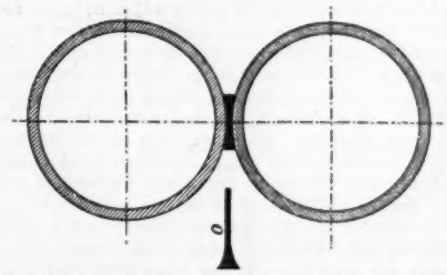


Fig. 4. Copper Calking Strip Between Firebox Tubes.

These nipples, in the first boiler that was built, are welded to the tubes, the flange being riveted and brazed to the nipple. An advantageous variation of the connection, also described by the inventor, is to make each box-tube in two independent halves, the top end of each half being bent upward and expanded into the steam drum like an ordinary fire-tube; in this case, of course, the flat bottom of the steam drum is not used, but the drum is a simple riveted shell. The arrangement last described also permits of placing the connections for the tubes composing the rear wall of the firebox wholly within the latter.

As actually constructed, the connection of the rear end tubes is less simple. Fig. 5 shows the construction. The end wall tubes have nipples projecting upward and rearward, and bolted by a flange to the (perforated) rear wall of the enclosing shell. On the outside of the shell are bent copper connection tubes, which are bolted to the shell in register with these nipples and pass thence upward to a flange connection on the lower face of the steam drum. A small section of the rear shell wall, containing the nipple connections, is made removable for more convenient access to the nipples.

The lower ends of the box tubes are fixed into

the mud drum by expanding and beading. On account of the close spacing of the tubes the end of each is first drawn down to a smaller diameter, as shown in Fig. 6. To protect these connections from accumulations of ashes, flanged copper sleeves are fitted closely around the tubes at the joints resting on the mud drum and extending up to the level of the grate.

The front end of the mud drum communicates with the barrel of the boiler by two large copper or steel pipe connections (shown in Fig. 2), one at the forward end of each leg of the horseshoe-shaped mud drum. An alternative detail for this water connection is to continue the mud drum across the front of the horseshoe, carrying a number of smaller (3 to 4-in.) tubes from the top of this part of the mud drum upward to the boiler shell (Fig. 7). This latter arrangement is preferable when the general design of the locomotive does not give room for the large elbow connections shown in Fig. 2.

The method of fastening the elements of the firebox together may be understood by reference to Fig. 5. At about the level of the center of the fire-door each box-tube carries a small clip or lug A, brazed to the tube, by which adjacent tubes are bolted together; the front and rear ones of these lugs are also bolted to the fire-door ring and to the front wall of the enclosing shell, respectively. Above this row of lugs are one or more straps of flat iron, B in Fig. 5, which are fastened to the fire-door ring and the front wall of the firebox shell. Other similar straps C, located below the belt of lugs, fasten to the inner element of the rear firebox wall instead of the fire-door ring.

To support the firebox, as already noted, four hangers are provided, one at each corner, which fasten the mud drum to the angle-iron edging of the enclosing shell. The shell being supported on the engine frame and the box tubes on the mud drum, the whole firebox structure is thus supported. In the first boiler built, these hangers were not fastened directly to the enclosing shell; but bore each on a helical spring resting on the angle-door of the shell. This was intended to provide an elastic support for the structure, but as the box tubes have a rigid connection at the top to the rear extension of the steam drum, it is thought that the springs are not necessary. It may be mentioned that a brick fire arch is provided in the firebox, as indicated by dotted lines in Fig. 2. This arch is carried by brackets attached to the front wall of the enclosing shell and at its forward end connects with the brick wall which forms the front of the firebox under the barrel of the boiler.

A locomotive fitted with this type of boiler was put into service on the Austrian State Railways in 1901, and used successively in switching, helping, and freight-hauling service. The results are said to have been excellent, and a number of other engines were ordered to be fitted with this boiler. The principal features of advantage which the Brotan boiler presents in comparison with the ordinary locomotive boiler, are the absence of stay bolted flat surfaces, greater heating surface with thinner firebox walls, less liability to damage in the firebox and greater ease of repair when damaged, better circulation, the ability to carry higher pressure with equal safety, and, as is said to have been shown by comparative results in actual use, greater steaming power and decreased fuel consumption as compared with the same engine having the usual type of boiler. From a table of dimensions of a locomotive weighing 45 tons (without tender but with full boiler) the Brotan boiler and firebox appears to weigh about 5% more than the ordinary boiler and firebox; however, it is able to carry 160 lbs. against 130 lbs. in the old boiler. The cost of the new boiler is said to be less than that of the old type.

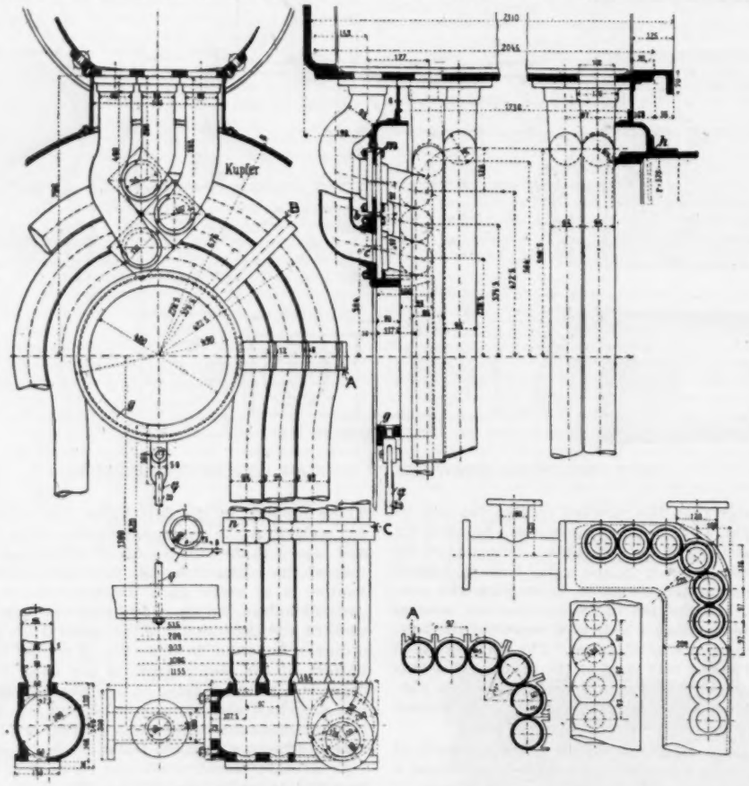



FIG. 5. DETAILS OF REAR END OF FIREBOX AND MUD DRUM.

## COMMERCIAL WROUGHT IRON AND STEEL PIPE.*

By Franklin Riffle, M. Am. Soc. C. E.†

The development of the wrought-pipe industry forms one of the great industrial achievements of the nineteenth century. It had its beginning about 100 years ago, when the introduction of illuminating gas in England created a demand for pipe which was promptly met by the manufacture of gun barrels. The close of the long wars between England and France left on the hands of manufacturers large stocks of this commodity. These were converted into gas pipes by threading the large ends on the inside, and the small ends on the outside. They were easily connected by screwing the small end of one tube into the large end of another.

The gas pipes of 100 years ago were lap welded by hand over a bar or mandrel. The process was slow and laborious. The several steps were as follows:

1. Preparing a strip of iron of the required width, thickness and length.
2. Scarfing the longitudinal edges.
3. Heating to the proper temperature for bending.
4. Bending or rounding over a mandrel until the scarfed edges lapped.

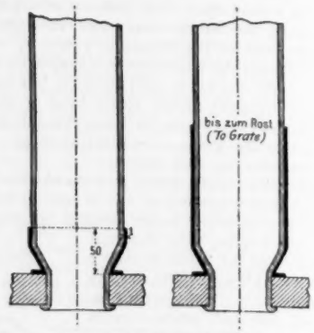



Fig. 6. Connection of Box-Tubes to Mud Drum.

5. Heating in a forge, a few inches at a time, to a welding temperature.
6. Slipping over a bar, and then hammering the lapping edges until welded. These last two operations had to be repeated many times before the tube was completed.

The first decided advance in the manufacture of gas pipe was made in 1825, when James Russell, a manufacturer of gas tubes and gun barrels, discovered that a satisfactory weld could be made by butting and squeezing together the edges of a properly rounded and heated strip of iron. This was accomplished by means of a pair of semi-circular dies, slightly smaller in circumference than the width of the metal. As in the lap-weld process, the butt-welded tube was made by alternately heating and welding a few inches at a time.

Coincident with the invention of the butt-weld process of making pipe, Mr. Russell introduced the screw socket or coupling. This was an important step in the evolution of wrought pipe, since it permitted the sections to be made of uniform diameter from end to end.

John Russell's method of butt-welding pipe was short-lived. It was soon supplanted by an improved process which consisted of drawing the prepared strip of metal, properly heated, through a circular die, thus welding the entire tube at one operation. This is the method of manufacturing butt-welded pipe at the present day, although, of course, many economic improvements have been made since its introduction 75 years ago.

The butt-weld process involves the following operations:

1. Clipping the corners of one end of a strip of metal, previously rolled to the required dimensions.
2. Heating to a welding temperature in the welding furnace.
3. Seizing the clipped end and drawing from the furnace with a pair of tongs, over which the operator throws a die or "bell" which drops into the bell holder; then connecting the handle of the tongs by means of a "hook" of special design, to a link of a moving chain which draws the strip through the "bell," thus welding it into pipe.
4. Delivering to a moving table, on which the pipe partially cools.
5. Feeding into sizing rolls, which shape and size to the required diameter.
6. Passing through straightening rolls.
7. Delivering to a slow-motion shifting rack and cooling.
8. Cutting off the ragged ends and threading.
9. Screwing on couplings.
10. Testing by hydrostatic pressure.

With the development of steam engineering during the second quarter of the nineteenth century, came a demand for pipe of greater strength than the butt-welded pipe of that period. As the hand method of lapwelding was slow and expensive, the feasibility of making lap-welded pipe by machinery naturally occupied the attention of the more progressive pipe manufacturers. After a long period of experimenting, Messrs. Russell had the satisfaction of put-

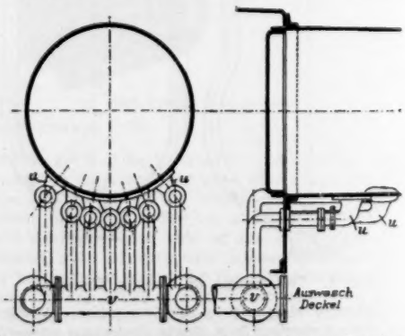



Fig. 7. Alternative Water Connection at Front of Mud Drum.

*A paper read before the eleventh annual meeting of the Pacific Coast Gas Association.

†Manager Iron and Pipe Departments, Dunham, Carrigan & Hayden Co., San Francisco, Cal.

ting in successful operation in 1835, the process of lap-welding which (although somewhat modified and much improved) is used by the pipe mills of the present day. The operations are briefly as follows:

1. Beveling the edges of a strip of metal of the required dimensions.
2. Charging into the bending furnace.
3. Drawing from the furnace at a red heat through a die and over a mandrel, thus rounding the metal and overlapping the edges. (For pipes of large diameters it is necessary to use bending rolls, similar to those used in boiler shops.)
4. Charging into the welding furnace.
5. Pushing from the furnace by means of a long rod, termed the "push bar," into the welding rolls. These rolls are grooved to correspond with the outside diameter of the pipe. A projectile-shaped mandrel held in position by a long bar, corresponds with the inside diameter of the pipe. As the pipe is drawn over the mandrel the rolls exert sufficient pressure on the laps to weld them. Each pipe as fast as welded is quickly disengaged by withdrawing the bar from the mandrel.

The finishing operations are the same as in the butt-weld process.

The great essential in the manufacture of welded pipe is a properly designed welding furnace, in which the skelp can be brought to a uniform welding temperature throughout its entire length. Next in importance is the personal factor. It is one man's business to see that the skelp is heated precisely to the proper point before it is taken from the furnace. If the temperature is below this point an imperfect weld results, and if over heated the metal is burned or hardened. It is another man's duty to inspect the pipe the moment it leaves the rolls, and to decide instantly and correctly whether the weld is perfect or otherwise. If imperfect the pipe is returned to the welding furnace, and is again passed through the welding rolls. These are responsible positions, and men who can fill them satisfactorily need not be out of employment.

Welded pipe is made as small as ⅛-in. inside diameter, and as large as 30-in. outside diameter. There is but one mill in the United States prepared to make the latter size. This mill (which is now arranging to rebuild its entire plant at a cost of $10,000,000) contemplates making pipe of 36 ins. outside diameter as soon as the necessary equipment can be installed.

The minimum thickness of metal used in making lap-welded pipe is determined by the capacity of the tube to resist collapse when heated to the welding temperature. Standard casing and boiler tubes are close, if not quite, to the minimum thickness.

## LAP-WELDING VS. BUTT-WELDING.

For many years the butt-weld process of making pipe was confined to the very small sizes, 1¼ ins. being the maximum. During recent years, however, the process has been extended to include larger sizes. At the present time it is the common practice among the pipe mills of the United States to make 3-in. pipe, and all smaller sizes by the butt-weld process, and larger sizes by the lap-weld process. This statement applies only to standard pipe, since the lighter classes of tubular goods can be made only by lap-welding. The strength of a butt-weld is dependent largely upon the thickness of the metal, and it would seem that the thicknesses adopted for standard pipe are very close to the minimum thicknesses that can be relied upon to give good results.

While it may be possible to make butt-weld pipe larger than 3 ins. in diameter the cost would greatly exceed that of pipe made by the lap-weld process, and therefore manufacturers have fixed upon 3-in. pipe as the maximum butt-weld size, purely for economic reasons. The writer recently visited a mill where 3-in. standard pipe was being made by both processes, and was informed by the superintendent that there is no appreciable difference in the relative cost of manufacture for this particular size.

The question is sometimes asked, "In what respect, if any, is lap-welded pipe superior to butt-welded pipe?" Manufacturers of exclusively butt-weld sizes of pipe assert that butt-welded pipe stands the mill hydrostatic test as successfully as lap-welded pipe, which is perhaps true. The severer test of actual use, however, under varied conditions has established beyond a question the superiority of lap-welded pipe. Butt-welded pipe may give equally good service when the conditions attending its use are not unusual, but it should not be used for important pipe lines, if for no other reason than for its tendency to open at the weld when subjected to the excessive torsional or bending stresses which cannot be wholly eliminated in the work of laying, especially where tight joints are an important consideration. Therefore, because of its reliability for work of this character, manufacturers use only the lap-welded process in making line pipe.

## PIPE JOINTS.

With reference to the manner of joining the lengths or sections, wrought pipe may be classified as follows:

1. Screw Joint Pipe, having threaded ends, which usually screw into connecting sockets.
2. Flange Joint Pipe, having flanges fastened to the ends by one of four general methods, viz: (a) shrunk on, then beaded and expanded; (b) screwed; (c) riveted; (d) welded.

With reference to weight, or thickness of shell, screw-joint pipe may be divided into four classes of standards:

1. Embracing standard pipe, line pipe, drive pipe, and tubing.
2. Extra strong pipe.
3. Double extra strong pipe.
4. Casing.

Taking the weight or thickness of Class 1 as a standard of comparison and representing it by 1, then extra strong pipe is approximately 1 4-10, double extra strong pipe 2 8-10, and standard casing 6-10. Or if standard casing is represented by 1, standard pipe is approximately 1 2-3, extra strong pipe 2 1-3 and double extra strong pipe, 4 2-3. In other words, standard pipe is nearly twice the thickness of standard casing, and about 1-3 the thickness of double extra strong pipe, of the same diameter. Classes 1, 2 and 3 have the same outside diameter.

The standard pipe of commerce, commonly known as merchant pipe, is of lighter weight than standard weight or full-weight pipe. Manufacturers specify that the latter may have a permissible variation of 5% above or 5% below the nominal or table weights. Merchant pipe, however, such as manufacturers and jobbers carry in stock, is almost invariably under the nominal weight. The actual weight varies somewhat among the different mills, but usually lies between the limits of 5% and 10% below the table weight. Line pipe, drive pipe, and tubing are full weight, subject to the "permissible variation" of the manufacturer. The same is true of extra strong and double extra strong pipe.

Drive pipe is used chiefly in oil wells, and is designed to stand heavy driving. The ends therefore are turned in a lathe, in order that they may meet at the centers of the couplings to form a continuous column, which can be driven without damage to the threads.

As tubing is used in wells for pump barrels it is both plugged and reamed to ensure a smooth interior.

The smaller sizes of double extra strong pipe are made by driving one piece of extra strong pipe inside of another. The larger sizes, however, are made in the usual way.

Standard casing is the lightest screw joint pipe that can be made. Casing which is made heavier than the standard is termed "special" casing. The increased thickness reduces the interior diameter. Thus the outside diameter of 5-in. standard casing is 5.25 ins., and the inside diameter is 4.95 ins. The nominal weight is 8.20 lbs. per ft. Three "special" weights of this size are made, weighing respectively 9.86, 12.80 and 15.88 lbs. per ft. The corresponding inside diameters are 4.87, 4.75, and 4.65 ins., the outside diameter remaining unchanged. This arrangement permits the use of standard casing couplings and fittings for all the special weights of casing having the same nominal diameter.

Oil-well casing differs from irrigation casing in having longer and heavier couplings. It is also subjected at the mill to more rigid inspection, as well as to a higher hydrostatic test. Casing is sometimes made to order with inserted joints, by expanding and threading the inside of one end, and the outside of the other end.

"Flush joint casing" is also used on rare occasions.—This joint is made by removing with a lathe half the thickness of the shell from the outside of the pipe at one end, and from the inside at the other end, and then cutting male and female threads on the respective ends.

## PIPE THREADS.

Pipe threads are designated by the number of threads per inch.

There are 22 sizes of standard pipe between ¼-in. and 12 ins., inclusive. As the pitch of the standard pipe thread is approximately the same as its depth, and as pipes of different diameters vary in thickness of shell, it is apparent that the same number of threads per inch cannot well be used for all sizes of pipe. However, only five different pitches are used for the 22 sizes, and 21 thicknesses of standard pipe.

Taking 4-in. standard pipe as an illustration, the standard thread has a pitch of ¼-in. (8 threads per inch) and is 1.8 ins. long. It consists of 8 perfect V-threads, then 2 threads that are perfect at the bottom, and slightly flattened on the top, and finally 4 imperfect threads, the last of which is but little more than a scratch. This is often referred to as a vanishing thread.

Casing threads are considerably finer than standard pipe threads, 4-in. casing, for example, having 14 threads per inch.

The standard pipe thread has a taper of ⅜-in. per ft. Drive pipe threads are practically straight. The threads on line pipe, tubing and casings have a taper of ¾-in. per ft., and the coupling threads are correspondingly tapered from each end of the coupling to its center. The threads of standard and drive pipe couplings are straight.

Couplings for oil country tubular goods (drive pipe, line pipe, tubing and casing) and hydraulic pipe (extra and double extra strong) are recessed at the ends.

When the pipes are properly screwed together, the recessed ends of the couplings extend beyond the threads, and fit snugly to the pipes, thus preventing any bending stresses from reaching the threads. By means of the taper threads on both the pipe and the coupling, a perfect contact is secured for every thread. These two features (the protective sleeve and taper threads) combine to form a joint of maximum strength and tightness.

In threading pipes and couplings the mills work closely to the standards for the several classes and diameters. Excepting the smaller sizes of merchant pipe it is customary among the best mills to screw a thread protector on the exposed thread of each length of pipe, especially for long-distance transportation. Quite a variety of thread protectors are in use, some made of light hoop iron, corrugated to fit the threads, and some of band iron; but for transportation across the continent to the Pacific Coast only the heavy, half-coupling protector has proven satisfactory.

The fact cannot be emphasized too strongly that approximately perfect threads are a prime essential for making absolutely tight joints in a pipe line. Many leaky screw joints are due to a disregard of this fact. In cutting threads in the field, and even in pipe-fitting shops, mill standards are too often ignored. In some cases the perfect threads are quite imperfect, while in other cases some are entirely lacking. Threads of this character are very liable to make troublesome joints.

On the other hand, Mr. R. T. Crane, of The Crane Co., Chicago, holds that pipe threads may be quite badly damaged without seriously impairing their efficiency. He has published the following experiment:

> An 8-in. coupling and a piece of 8-in. pipe were taken and throughout the entire length of the bearing of the thread in the coupling and on the pipe were made grooves such as are likely to occur when there is a defective weld. The grooves extended about half the depth of the thread, and were made an inch apart for the entire circumference of the pipe. The threads were flattened both on the pipe and in the coupling, except a mere speck in each. Also to offset the broken thread feature of the question, three grooves were cut in the pipe ¼-in. wide and the full depth of the thread. This coupling and piece of pipe were then screwed together, and tested with 500 lbs. of air, at which pressure the joint was tight, demonstrating conclusively the stupidity of steamfitters and engineers on this subject. . . . The amount of defect on the threads used in this test was at least 100% greater than that for which the regular steamfitter or engineer will reject material.

The writer believes that Mr. Crane has over-estimated the value of this experiment. It is far from being conclusive. It proves only the possibility of making a tight joint with a thread that is far from being perfect. And while it may serve to show the unreasonableness of condemning threads that are not quite up to the standard of excellence, it does not prove that damaged threads are always to be relied upon to make tight joints. This is especially true of a pipe line, for in the field there are no such facilities for effective joint making as Mr. Crane evidently had at hand when making his experiment. It is not an easy matter to lay a tight pipe line under the most favorable conditions, although the chances of success are unquestionably better when the threads are in perfect condition than when they are crudely cut, or badly damaged.

The holding power, or shearing strength, of a screw-joint is much greater than is generally supposed. When all the threads have the maximum contact (as in taper-thread couplings), their combined stripping strength is several times the ultimate tensile strength of the shell of the pipe. This statement applies no less to the fine threads of casing than to the coarse threads of line pipe; but it does not apply to the ordinary straight-thread couplings, such as are used on merchant pipe, since in practice the contact area of the threads, is somewhat less than in taper-thread couplings.

Taking for illustration, 2-in. line pipe, and assuming the threads to be in perfect contact, the total contact area approximates 5 sq. ins., while the metal area of the pipe is 1.07 sq. ins. Assuming the tensile strength of the metal to be 60,000 lbs., and the shearing strength to be 45,000 lbs. per sq. in., then the ultimate strength of the pipe is 64,200 lbs., while the stripping strength of the joint is 225,000 lbs. This leaves a wide margin of safety for straight-thread couplings, imperfect threads, or carelessness in making the joints.

The writer is indebted to a representative of a leading pipe mill for the following description of a series of tests that were made for the purpose of determining the relative strength of taper-thread line pipe couplings, and standard, or straight-thread couplings:

> TEST NO. 1.—A piece of 2-in. standard pipe was screwed into a line pipe coupling, and when subjected to a tensile strain, broke some distance from the coupling at 65,000 lbs., equivalent to about 60,500 lbs. per sq. in. of metal area. After this test all the pipe and coupling threads were found to be in perfect condition.
>
> TEST NO. 2.—A piece of 2-in. standard pipe (cut from the same length as the piece used in Test No. 1) was screwed into a standard coupling. It broke in the socket under a tensile strain of 37,000 lbs.
>
> This test was repeated, with the result that the pipe was pulled out of the socket by stripping the few threads that were in contact.
>
> TEST NO. 3.—A 2-in. line pipe coupling was screwed on a piece of pipe by machinery, and the coupling then taken off. This operation was repeated 160 times, after which an examination showed that all the pipe and coupling threads were in as good condition as before the test.
>
> TEST NO. 4.—The same test as Test No. 3 was applied to a piece of 2-in. pipe and a standard coupling; but while screwing on the coupling the tenth time the pipe broke inside the coupling.

These tests demonstrated that by using taper-thread coupling the joints can be made stronger than the shell of the pipe, while joints made of standard or straight-thread couplings are the weak portions of a pipe line.

## STEEL PIPE VS. WROUGHT-IRON PIPE.

The crowning achiements of the pipe industry has been the successful use of steel in the manufacture of wrought pipe. This innovation by one of the leading pipe mills in the early '80's, met with a storm of opposition, not only from rival mills, but from engineers as well.

It was represented that a perfect weld could not be made with steel; that steel pipe could not be threaded, and that it was even more unreliable and treacherous when under

pressure than cast-iron pipe. As late as ten years ago, many engineers refused to accept steel pipe as a substitute for wrought-iron pipe, although manufacturers were prepared then, as now, to prove its superiority. Since then engineers have so universally adapted themselves to the new conditions, that when an occasional non-conformist specifies wrought-iron pipe, in preference to steel, it is assumed that he has not kept pace with the wonderful progress that has been made in the manufacture of steel tubular goods during the last decade. At the present time manufacturers are producing a uniform quality of steel for pipes, that for malleability, ductility and weldability, cannot be excelled by the more expensive product of the puddling furnace, while for tensile strength it is vastly superior to the highest grades of iron.

Many persons are deceived by the high-sounding phrase, "Guaranteed Wrought-Iron Pipe." This class of pipe is seldom made of puddled iron. On the contrary, it is oftener made of re-rolled scrap, which contains a varying percentage of steel. If any one desires to satisfy himself regarding the sort of product that results from re-rolling a mixture of iron and steel scrap, such as is collected by junk dealers, let him inspect the fracture of an average bar of so-called "refined" iron. He will probably discover that a considerable portion of the area is crystalline in structure, and that the steel crystals and iron fiber have little or no affinity. The same character of material is largely used in the manufacture of the "guaranteed wrought-iron pipe," which is sold to the unsophisticated, at a price approximately 10% higher than that of the best quality of steel pipe.

In 1897 Prof. Henry M. Howe, an eminent authority on the metallurgy of steel, made a series of experiments for the purpose of determining the relative bursting strength, tensile strength and frictional resistance of wrought-iron and steel pipe. The information derived from these experiments is probably the most reliable and conclusive that has ever been compiled, relative to the comparative merits of iron and steel pipe.

Three classes of wrought-iron and steel pipe were selected, viz.: 2-in. line pipe, 2-in. tubing, and 5⅝-in. casing. The wrought-iron pipes were obtained from three manufacturers of good reputation, while the steel pipes were provided by the manufacturer who engaged Professor Howe to make the experiments. Neither the wrought-iron nor the steel pipes were made especially for these tests. In his report, Professor Howe states that he selected the 2-in. steel tubing from a large pile at the shipping platform of the mill, and the 2-in. steel line pipe from two large piles at the storehouse, each about 12 ft. high and 60 pipes wide. The 5⅝-in steel casing was selected partly from a pile at the shipping platform, partly from another pile at the mill, and partly as it came from the welding rolls. Professor Howe took the additional precaution to mark each pipe selected with his private stamp.

BURSTING TESTS.—The pipes which were selected for the bursting tests were provided with welded heads, after which they were subjected to hydrostatic pressure until they burst, the bursting pressures being determined by a gage. The highest pressure available was 6,000 lbs. per square inch. All of the wrought iron pipes burst below this pressure, while approximately two-thirds of the 2-in. steel tubing, and one-third of the 2-in. line pipes successfully resisted the maximum pressure.

The following table shows in condensed form the average results of the experiments:

Bursting Tests by Hydrostatic Pressure.

| | Bursting pressure, lbs. per sq. in | | |
|---|---|---|---|
| | 2-in. line pipe. | 2-in. tubing. | 5⅝-in. casing. |
| Wrought iron, minimum | 1,000 | 3,300 | 250 |
| Steel, minimum | 2,300 | 5,150 | 1,450 |
| Wrought iron, maximum | 4,000 | 5,000 | 1,400 |
| Steel, maximum | 6,000 | 6,000 | 2,750 |
| Wrought-iron pipe tests, av'ge. | 2,918 | 4,106 | 931 |
| Steel, average of all pipes tested | 4,733 | 5,800 | 2,038 |

A comparison of the average results shows that the steel pipes excelled the wrought-iron pipes by 62% in the line pipe, 41% in the tubing, and 119% in the casing.

TENSILE TESTS.—For making the tensile tests short pieces were cut from certain of the steel and wrought-iron pipes. These were accurately threaded and fitted with special extra heavy couplings. They were then pulled lengthwise in a tensile testing machine until they tore in two. The stress was noted, and reduced to pounds per square inch of metal area. The average results are shown in tabulated form as follows:

| | Tensile strength, lbs. per sq in. of net section | | |
|---|---|---|---|
| | 2-in. line pipe. | 2-in. tubing. | 5⅝-in. casing. |
| Wrought iron, minimum | 43,107 | 47,244 | 47,312 |
| Steel, minimum | 63,023 | 60 370 | 75,931 |
| Wrought iron, maximum | 53 809 | 55 074 | 61,309 |
| Steel, maximum | 67,598 | 66,495 | 91,791 |
| Wrought-iron pipe tests, av'ge | 50,002 | 51,852 | 54,311 |
| Steel, average of all pipes tested | 65,999 | 63,057 | 82,355 |

The table shows that the steel pipes excelled the wrought-iron pipes by the following percentages:

Line pipe, 32%; tubing, 22%; casing, 52%.

FRICTION TESTS.—Without going into details it may be stated briefly that the friction tests showed that the steel pipes offered less resistance to the flow of water than wrought iron. To quote Professor Howe: "For a given initial pressure the final pressure is here about 6% greater in case of steel than in that of wrought iron."

The concluding sentence of Professor Howe's report relative to the above series of tests is as follows:

To sum up, the steel pipes excel the wrought iron very greatly in bursting strength, very considerably in tensile strength, and somewhat in offering less frictional resistance to the flow of water.

THREADING STEEL PIPE.—The most common complaint against steel pipe is that it is more difficult to thread than wrought iron. This difficulty can be usually traced to the use of old, worn-out threading tools. Steel pipe is well threaded at the various pipe mills, and is equally well threaded in the best pipe fitting shops. In fact, no pipe fitter or pipe layer need have the slightest difficulty with threading steel pipe, if he will take the precaution to provide himself with up-to-date threading tools, and then keep them in good repair.

CORROSION.—Another objection that is sometimes urged against steel pipe is that it corrodes much more rapidly than wrought iron. Of late years this objection has been shown to be based on prejudice rather than fact. The writer is frequently told that wrought-iron pipe is much more durable than steel; but when he attempts to get at the facts, his informants invariably fail to furnish any satisfactory evidence in support of their extravagant claims. A mine superintendent recently made the astonishing statement that he knew by experience that wrought-iron pipe will last fully twice as long as steel pipe. A brief cross-examination developed the fact that he had used some light sheet iron pipe some 20 years ago in the state of Nevada, and had lated used sheet-steel pipe in California. He could not remember whether the thickness of metal was the same in both cases or not, and had no knowledge of the chemical properties of the water and soil of the two localities. All that he knew was, that the iron pipe lasted twice as long as the steel.

As early as 1895 Professor Howe, in a paper entitled "Relative Corrosion of Wrought Iron and Steel," summed up his analysis of a considerable number of experiments made by himself and other investigators, in the following language:

It is misleading to say either that normal low-carbon steel corrodes more or that it corrodes less than wrought iron, since the irrelative corrosion varies with the enclosing medium. I think we have very strong reason to believe that such steel, when carefully made, resists corrosion in fresh water as well as wrought iron; in cold sea water, at least nearly as well as wrought iron; and in acidulated water better than wrought iron.

Five years later Professor Howe published the results of a series of experiments in which he exposed a number of plates of wrought iron, steel, and nickel steel, ¼-in. thick, to the action of sea water, river water, and the weather, for two periods of one year each. The results are summed up in the following table:

| | Corrosive Tests. Sea water. | Fresh water. | Weather. | Average. |
|---|---|---|---|---|
| Wrought iron | 100 | 100 | 100 | 100 |
| Soft steel | 114 | 94 | 103 | 103 |
| 3% nickel steel | 83 | 80 | 67 | 77 |
| 26% nickel steel | 32 | 32 | 30 | 31 |

This table practically confirms Professor Howe's statement of 1895, and should go far toward convincing any one, who is open to conviction, that the difference in the rate of corrosion between iron and steel is hardly appreciable, and that wrought iron possesses no advantage over steel in this respect.

No matter whether we use wrought-iron or steel pipe, corrosion is a factor that cannot be ignored. Its rapidity of action depends to a very great extent on the chemical properties of the soil in which the pipe is laid. In some soils of California, light riveted pipe coated with asphaltum has done good service as water mains for 25 or 30 years, while in other soils the same class of pipe has lasted less than as many months. How best to protect iron and steel pipe from corrosion has long been a problem that has given rise to much research and experiment. More than 30 years ago the manufacturers of light-riveted pipe in California realized that the success of this class of pipe depended upon the application of satisfactory preservative coating. Judging from riveted pipes that were recently taken up in Oakland, after having been in the ground since the early '70's, it was customary in those days to apply the asphalt coating very generously and with great care. Since then manufacturers of riveted pipe, refiners of asphaltum, and engineers have devoted much time and thought to this all-important subject, and at the present day the coating of steel pipe with asphaltum is probably better understood in California than anywhere else in the world. It is possible to purchase in this market asphalt preparations for coating pipe that will give excellent results if properly used. And here it should be emphasized that in coating pipe with asphaltum, experience, good judgment and thoroughness are the prerequisites of success. Probably 90% of the failures of asphalt coating to preserve pipe from corrosion is due to the absence of one or all of these important qualifications.

In view of the importance of the subject the following points are suggested regarding the coating of pipe with asphaltum:

1. The pipe should be thoroughly cleaned. Every particle of rust, and as much of the mill scale as possible, should be removed.
2. The asphalt bath should be brought to a temperature of 300° F. Its proper consistency may be determined by dipping in the mixture a strip of thin steel, and then withdrawing and plunging into cold water. If after this treatment the coating is soft and sticky the mixture contains too large a proportion of the fluxing or liquid grade of asphalt, and if in bending the strip the coating shows a tendency to crack and flake off there is an excess of the solid grade.
3. The pipe should be left in the bath until it has had sufficient time to attain the same temperature as the asphaltum. For the heavy types of welded pipe, it would be advisable to heat each pipe to the same temperature as the bath before immersing.
4. As a thick coating of asphaltum affords more thorough protection than a thin coating, two dippings are recommended, in which case the first coating should be allowed to harden before re-dipping.
5. Pipe should be coated in the vicinity of the laying. Under no circumstances should it be dipped before leaving the mill, except as a precaution against rusting en-route, in which case it should be re-dipped at destination. After the pipes receive their final coat the greatest care should be exercised to prevent abrasion, both in distributing along the trench and in laying. As a final precaution every pipe should be rigidly inspected for abrasions.

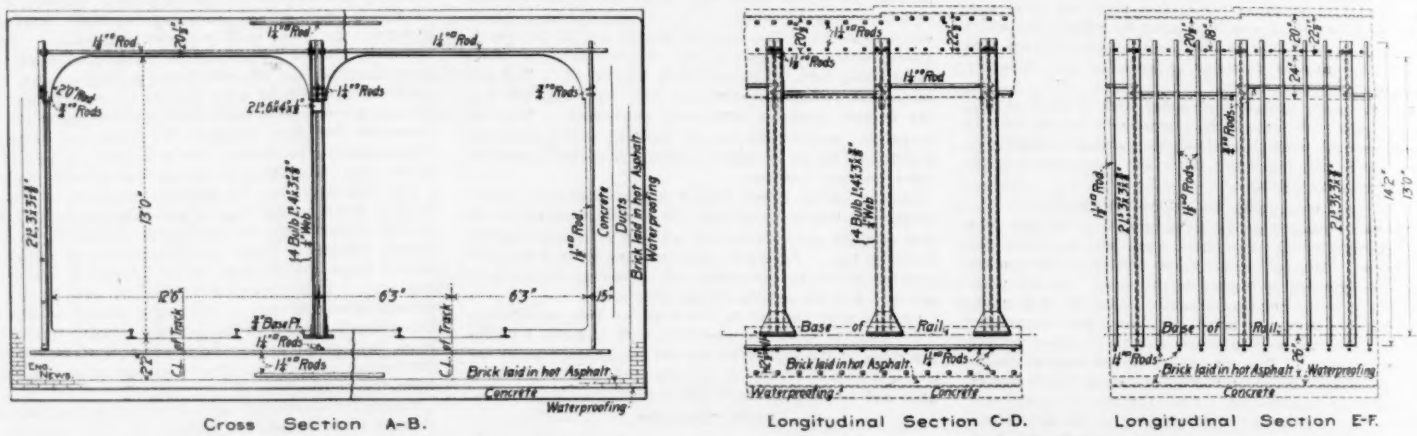



FIG. 103. TRANSVERSE AND LONGITUDINAL SECTIONS OF REINFORCED CONCRETE SUBWAY.

and wherever exposed, the metal should be given two coats of some reputable brand of asphalt paint.

The Sabin pipe coating ("a varnish enamel which is baked on at about 400° F.") has been used during the past ten years on several important steel pipe lines of large diameters, with excellent results. This process, however, requires an expensive plant, and for that reason is used only in connection with large and important undertakings.

The galvanizing and kalamein processes consist of immersing the pipe in a molten alloy, of which zinc is the chief ingredient in the former process, and tin in the latter.

Any one of the coatings mentioned, if properly applied, will add very materially to the life of steel pipe. But no known preservative, metallic or varnish, can be relied upon to successfully protect steel pipe from corrosion under all conditions of soil and water. Chemists may some day discover a universal remedy that will prove inexpensive and efficacious, but until then we must do the best we can with the methods now in vogue.

### SPECIFICATIONS.

Engineers sometimes prepare elaborate specifications for pipe without stopping to consider if they are practicable. So far as the quality of the material and the details of manufacture are concerned, it is always advisable to conform to the practice of the best pipe mills. All grades of steel, for example, are not equally weldable, therefore it would be a waste of time to specify steel that could not be welded. It is always a good plan to obtain from some reputable mill such specifications as are best adapted to the work in hand. Arrangement can then be made to have the pipe made in accordance with these specifications under the supervision of a competent inspector.

So far as testing the pipe at the mill is concerned, the standard hydrostatic tests for the several classes of pipe will answer, unless extraordinary working pressures are contemplated. The standard tests are as follows:

| | |
|---|---|
| Standard, Ex. strong and double Ex. strong pipe— | |
| 2-in. and smaller | 300 lbs. per. sq. in. |
| 2½-in. and larger | 500 " " " " |
| Line pipe: 6-in. and smaller | 1,500 " " " " |
| 7-in. to 12-in. | 800 " " " " |
| Tubing | 2,000 " " " " |
| Irrigation casing: 5⅝-in & smaller. | 500 " " " " |
| Larger sizes | 300 " " " " |
| Oil-well casing: 5⅝-in. & smaller. | 600 " " " " |
| Larger sizes | 500 " " " " |

Unless otherwise specified the several classes of pipe are tested at the mill to the above pressures. Arrangements can always be made for special tests when the pipe is to be used under high pressures. In no case should the mill test be less than 1½ times the proposed static pressure.

SPECIFICATIONS FOR MERCHANT PIPE.—The following specifications are sufficient to secure a good quality of merchant pipe at the ruling market price:

Pipe to be made of good, ductile metal of uniform quality, and to be straight and smooth. Threads to be clean cut in accordance with Manufacturers' Standard. Couplings to be well screwed on. Jointers not to exceed 8% of the total number of lengths. All pipe to be subjected to the standard hydrostatic test before leaving the mill.

SPECIFICATIONS FOR LINE PIPE.—Pipe to be made of steel of tough, ductile and uniform quality. Each length to be fitted with a full weight line-pipe coupling, the ends of which shall be recessed. The threads of the coupling to taper uniformly from both ends to the center. A half-coupling thread protector to be securely screwed on to the exposed end of each length of pipe. None of the pipe to vary more than 5% above or 5% below the standard weights. Jointers not to exceed 6% of the total number of lengths. Each length of pipe to be tested at the mill to a specified hydrostatic pressure.

If the pipe is for a high working pressure it would be advisable to specify the physical properties of the material.

MANUFACTURERS' LIMIT OF GUARANTEE.—Pipe manufacturers are usually willing to replace pipes that are clearly defective, but under no circumstances will they assume responsibility for damages, beyond the price of the pipe. Pipes are used for such a multiplicity of purposes and under such a variety of conditions, that manufacturers have been compelled, as a protective measure, to adopt the rule that "no charges for labor, or expenses required to repair defective goods, or occasioned by them, will be allowed."

The necessity for the adoption of such a rule suggests the fact that serious failures now and then occur in the use of pipe. Such failures, however, are rare, and their occurrence can be made exceedingly improbable, if they cannot entirely be prevented, by employing at the outset the services of a competent engineer—one who knows how to secure at a minimum cost the pipe best adapted to the requirements.

In conclusion the writer takes pleasure in acknowledging his indebtedness to Mr. Chas. M. Woods, Manager of Sales, and Mr. Thos. W. Brooks, Asst. Manager of Sales, of the National Tube Co. for the Pacific Coast, for having placed at his disposal certain records of tests made by the National Tube Co., of which he has freely availed himself in preparing this paper.

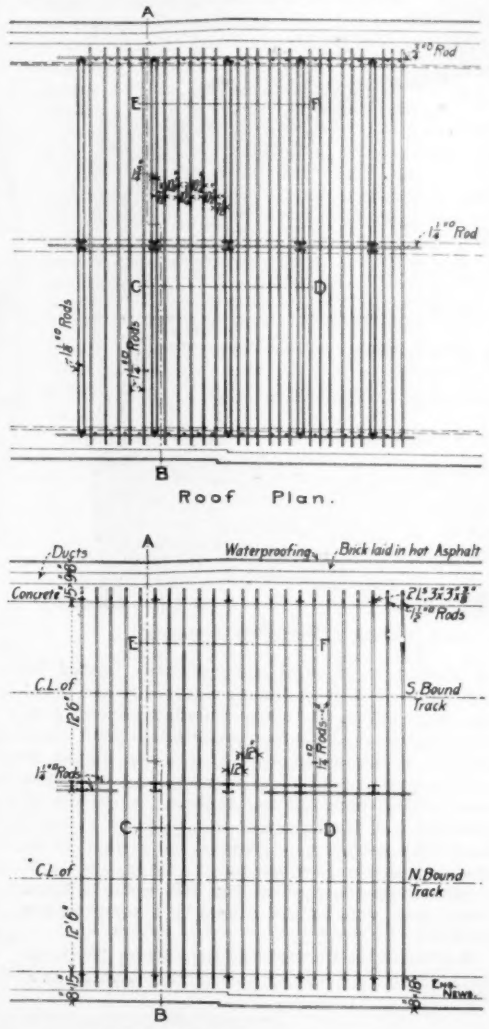



Fig. 104. Roof and Foundation Plans of Reinforced Concrete Subway.

## THE NEW YORK RAPID TRANSIT RAILWAY.

### XXIII.

### Contract Section IX-a: Land Section.

Contract Section IX-a of the Rapid Transit Railway extended from a point on Lenox Ave. 100 ft. north of 135th St., to and across the Harlem River, to the center line of Girard Ave., in the Borough of the Bronx. The subcontractors were McMullin & McBean, of New York city. The section was one of the most difficult to construct of any throughout the length of the subway, because of the tunnel under the Harlem River..

SUBWAY STRUCTURE.—The route of the subway included in Section IX-a follows the center line of Lenox Ave. to a point about midway between 141st and 142d streets, and then swings east and cuts under private property near 143d St. The line then passes under private property and the Harlem River until it strikes the line of 149th St., which it follows to the terminus of the section at Girard Ave. From 135th St. the tracks descend on a 0.35% grade to 139th St., then on a 0.9% grade to 141st St., and then they dip under the river on a 3% grade and rise with the same grade to a point beyond Girard Ave. on the opposite shore. From the curve at 141st St. a double-track branch line extends north along Lenox Ave. to the south bank of the Harlem River. This branch has a twofold purpose. It furnishes access to a car house and power plant, and enables trains to be made up at an intermediate point on the East Side line, and so relieve the pressure on the trains coming from the Bronx terminus.

Considering the main line first, the construction from 135th St. to 139th St. was standard double-track concrete and steel beam subway. From 139th St. to 141st St. a construction in concrete-steel was employed. The details of this concrete-steel subway are shown by Figs. 103 and 104, but before entering into an explanation of them the occasion for its development and use in place of the standard concrete and steel beam structure is desirable. In building the roof arches of the standard subway it was found that extreme care was necessary in tamping the concrete in place to ensure that it was crowded well up underneath the top flanges of the roof beams. To avoid this difficulty and the danger of undiscovered void spaces at the angle of the beam flanges and web, search was begun for a better form of construction. It was at first proposed that deck beams be used in place of I-beams for the roof construction, since by placing these beams with their bulbs uppermost the top flange angle so difficult of access in the I-beam construction would be done away with. Another line of study looking toward the total omission of the roof beams, and much of the other steel structure, and the substitution for them of a concrete-steel design, was pursued by Mr. St. John Clarke, General Inspector of Designs, and finally resulted in a design for substantially the construction shown by Figs. 103 and 104. This design was approved by the Chief Engineer after it had been carefully examined, and was ordered substituted for the steel-beam construction for all subway for which the metal had not already been contracted.

As will be seen from the drawings of Figs. 103 and 104, the general rectangular form of the standard concrete structure was retained in the new construction. Of the standard steel-bent framework the center bulb angle columns were retained, but the I-beam wall columns and the roof beams were replaced by other constructions. In place of the sidewall columns two angles riveted together to form a Tee set with its face toward the tracks, were substituted, and in place of each roof beam two 1¼-in. round rods were fastened between the bulb angle column and the sidewall Tee. Similar sets of tie rods connected the tops of the sidewall Tees and center columns from bent to bent longitudinally of the subway.

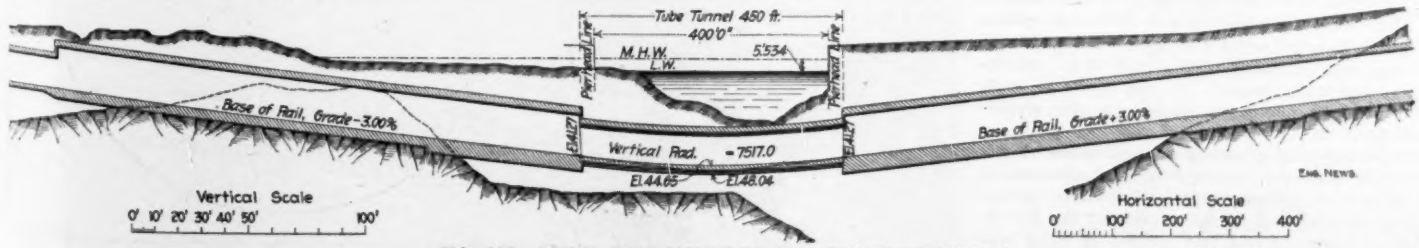



FIG. 105. PROFILE OF HARLEM RIVER TUNNEL SECTION.

In the standard construction concrete jack arches were molded between the sidewall columns and roof arches between the roof beams. The new construction replaced the jack arches with plane walls of concrete in which were embedded near the front surface twisted steel rods set vertically and 12 ins. apart. The succession of roof arches between beams was also replaced by a mono-

lithic plate molded to a curved intrados transversely over each track and flat on top, and containing a reinforcement of twisted steel rods. The roof reinforcement consisted of two sets of elements. Near the bottom of the roof plate there were parallel bars extending across the structure 12 ins. apart, the ends of which were wired to the tops of the sidewall rods, and near the top of the plate and over the center portion were shorter transverse rods of similar spacing. The floor of the subway in the new construction was reinforced in the same way as the roof, except that the shorter rods were placed at the bottom of the floor concrete and single longitudinal rods were placed under the bulb-angle columns.

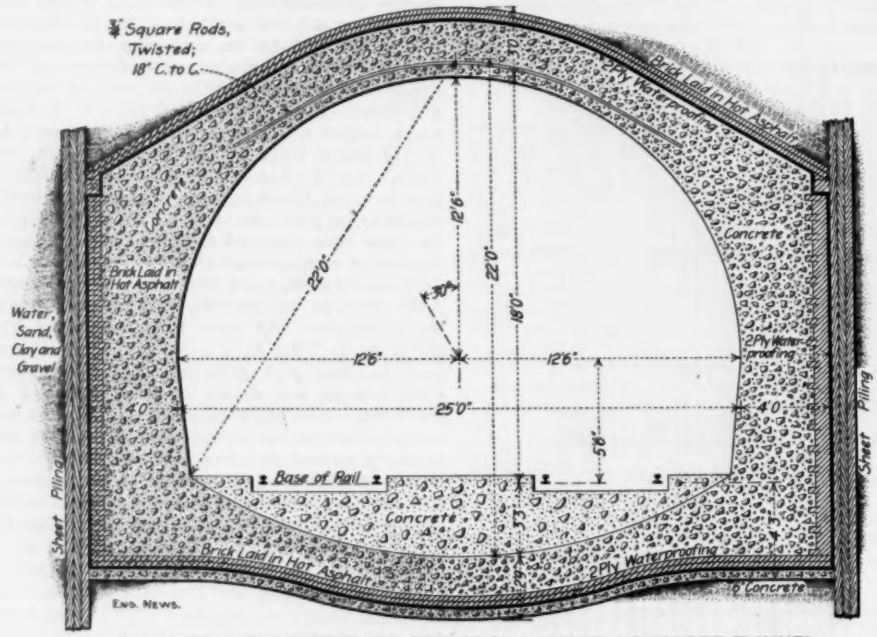



FIG. 106. TRANSVERSE SECTION OF DOUBLE TRACK CONCRETE TUNNEL.

Between 141st and 143d streets standard concrete and steel beam construction was used because of the long spans of the roof beams, rendered necessary by the widening of the subway at that point to allow the two tracks for the Bronx to swing off to the right, and the omission of some of the sulb angle columns because of the switches and crossovers. Some of these plate girder roof beams have a depth of 54 ins.

From 143d St. to the end of the section at Girard Ave., the subway structure was concrete arch construction, with the exception of 610 ft. under the Harlem River, which was double-tube tunnel of cast iron backed with concrete. Fig. 105 is a longitudinal section of the tunnel and its approaches. Fig. 106 is a transverse section of the concrete arch structure, and Fig. 107 is a transverse section of the double-tube construction. In explanation of Fig. 106, it should be said that the heavy construction shown was used for only a short length adjacent to each end of the tube construction. The profile, Fig. 105, shows approximately the extent of this heavy construction. Elsewhere the concrete arch, while retaining the same inside dimensions and sectional profile, had much thinner walls and omitted the twisted steel bar reinforcement of the roof arch. Structurally, the only feature of the concrete arch which need be noted particularly is the waterproofing.

This waterproofing, where it came below elevation +4, or ordinary high spring tide, consisted of two layers of asphalt paper, with coats of hot asphalt between and on the outside of the layers. Inside of this were laid two courses of brick laid in a hot asphalt composition filling both bed and collar joints. In order to prepare a proper bed for receiving the floor waterproofing upon the thin layer of concrete below it, it was found necessary to lay an additional layer of asphalt paper merely stuck at the laps with asphalt. Above elevation +4 the ordinary waterproofing was used.

Referring to the double-tube section, it will be noted that unlike most cast-iron lined tunnels which have been built in the past the lining is heavily backed with concrete, making an unusually heavy construction. The construction of the cast lining was also unusual, as is shown by Fig. 108. The segments had a 1-in. plate and 1-in. flanges on all four sides for joining them together. Each segment also had an intermediate circumferential flange to stiffen the plate. The connection of adjacent segments was formed by bolting the adjoining flanges together in the usual manner. Axially the connecting flanges were continuous, but the circumferential joints were staggered. All segments were heated and dipped in melted pitch previous to erection, and all joints were packed with felt and asphalt. A coating of concrete was placed inside the lining to a depth flush with the segment flanges. A bottom filling of concrete was provided for the track, and as is indicated by the profile, Fig. 105, a drain leads down each slope of the tunnel to a sump at the center.

MATERIALS PENETRATED.—From 135th St. to 143d St. the subsoil underlying Lenox Ave. consisted for a depth of 10 or 12 ft. of a filling of rocks, deposited when the avenue was graded, under which came sand containing some clay at the north end. From 143d St. to the end of the section at Girard Ave., the borings showed the following materials: At 143d St., coarse sand, gravel, coarse sand and rock; at high water shore line, mud 7 ft., coarse sand 4 ft., fine sand 1½ ft., and rock; at low water shore line, mud 12½ ft., sand 9 ft., sand and clay 30 ft.; at mid-river, mud 5 ft., clay 15 ft., sand 10 ft.; north side pier head, mud 9 ft., sand and clay 42½ ft., sand 2 ft., sand and clay 7½ ft.; at Exterior Ave., debris 12½ ft., clay 7½ ft., sand 7 ft., sand and clay 10 ft., sand 30 ft.; at River Ave., debris 12½ ft., sand and clay 7½ ft., sand 22 ft.; and at Girard Ave., debris 7½ ft., and sand and gravel 7½ ft. By reference to the profile, Fig. 105, it will be observed that the subway excavation encounters rock for a short distance on the south side of the river and again on the north side, about 600 ft. from the north pierhead line. Between these two points of rock the excavation is through mud, sand, gravel and clay, and as will be seen from Fig. 105, the depth of cover over the tunnel at the deepest part of the channel is very small.

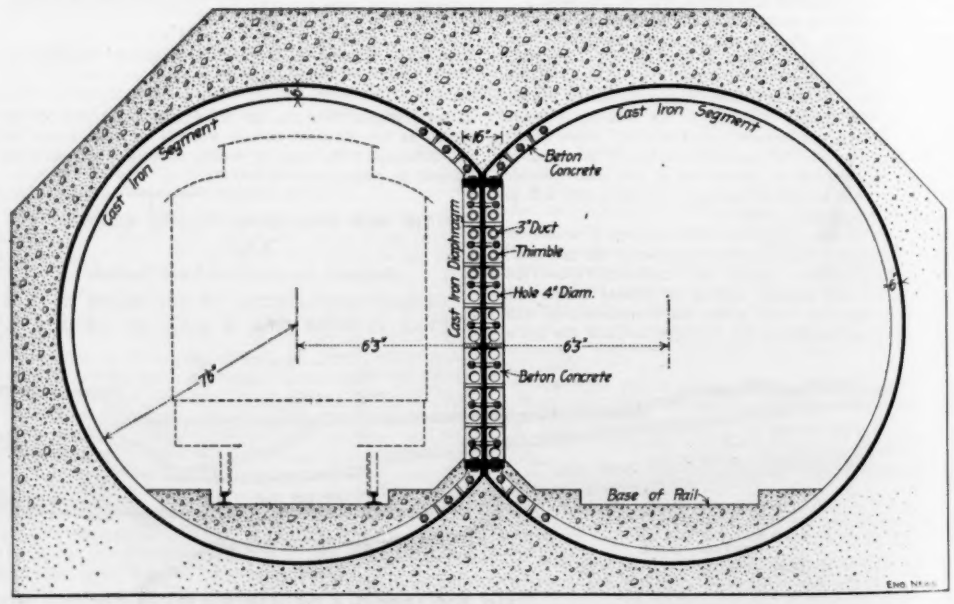



FIG. 107. TRANSVERSE SECTION OF TWIN TUBE TUNNEL.

STREET AND ADJACENT STRUCTURES.—A double-track street railway occupies the center of Lenox Ave., and its track structure had to be supported during construction. No other street structure along the section had to be specially considered until the north bank of the river was reached. Here the foundations of the 145th St. bridge approach came quite close to the excavation, and precaution was taken to sheet and brace the trench with unusual thoroughness, so that no shifting of the soil would occur.

METHODS OF WORK.—For sake of clearness and convenience in describing the methods of work on Contract Section IX-a, the work will be divided into two parts, which will be called the land sections and the river section. The bulk of the work comprised in the land sections was south of the Harlem River, and this will be considered first.

LAND SECTIONS.—On the east side of Lenox Ave., and bordering the Harlem River, there was an area of low land which was submerged at high water level, as is partially indicated by Fig. 105. This low land was secured by the contractor for a dumping ground, and this influenced largely the selection of the method of work which was adopted. Owing to the limited traffic on Lenox Ave., the contractor opened the subway excavation in a single wide trench. He began this trench at the dumping ground and carried it south along Lenox Ave., hauling the excavated material by locomotives and dump cars running on a track in the bottom of the trench. Fig. 109 shows a spoil train coming out of the trench on Lenox Ave., and turning east on the curve leading to the dumping grounds.

The trench along Lenox Ave. had to be sheeted and braced with considerable care because of the nature of the excavation, which, as previously stated, consisted of rock filling on sand, and because the edge of the trench came very close to

the southbound street car track. The method of timbering adopted consisted first in driving a row of piles along each side of the trench. These piles were spaced 10 ft. apart, and were from 30 to 35 ft. long and 12 ins. in diameter. After being driven the piles were cut off about 3 ft. below street grade. The piles of each row were then connected by inside waling timbers and cross braces

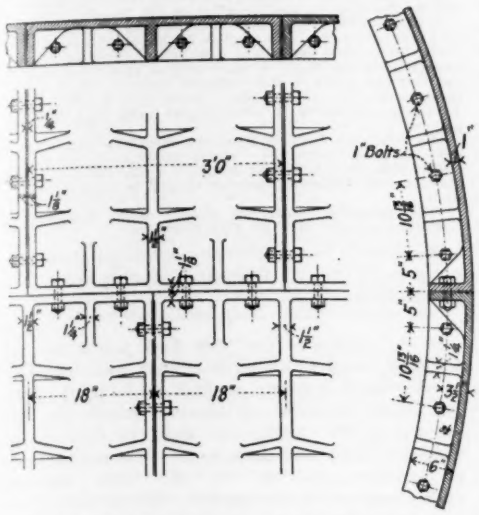



Fig. 108. Detail of Cast-Iron Lining for Twin Tube Tunnel.

were inserted between the opposite lines of waling. The sheeting was driven behind the waling timbers. It will be understood, of course, that the excavation and timbering proceeded simultaneously to a considerable extent. Practically the only machine of any note which was employed was a sort of gantry crane arrangement for handling the rocks encountered in the top part of the trench. This gantry straddled the trench and was moved along it on rollers. The trolley consisted of a platform car traveling on a track on the deck of the gantry and carrying a hand winch and tackle for lifting and lowering the pieces of rock. In operation this trolley raised the rocks from the side of the trench and swung them across and onto the dump cars. One of these gantries is shown in the distance in the view, Fig. 109.

Between 135th and 138th streets the introduction of a switch track made a three-track subway structure necessary, and this increased width brought the excavation underneath one of the street car tracks, which, consequently, had to be temporarily supported. The method of support adopted was to insert short planks under the track structure and to strut downward from them as the excavation deepened. The tracks were permanently supported during construction as soon as practicable upon transverse stringpieces of timber which were in turn suspended by metal straps from longitudinal timbers resting upon the tops of the above-mentioned piles.

A stretch of concrete and steel beam subway, complete with the exception of the roof arches, is shown by Fig. 110. The method of erecting this construction followed the usual practice. The subway floor was first placed and the column footings set; the steel bents were then erected, and finally the sidewalls and roof arches were molded in place. This general sequence of operations was also followed in building the concrete-steel structure. First, the floor concrete was placed with the reinforcement inserted at the proper points. The sidewall Tees, center columns and tie rods were then erected, and the sidewall concrete was deposited in suitable forms with the reinforcing bars inserted at the proper points. The roof centers were then erected, and the concrete with the embedded reinforcement was deposited on them.

The land sections of concrete arch subway were constructed in open cut and presented no uncommon features except the depth of excavation and the rigid trench timbering required by the soft ground, and on the north shore by the adjacent bridge foundations.

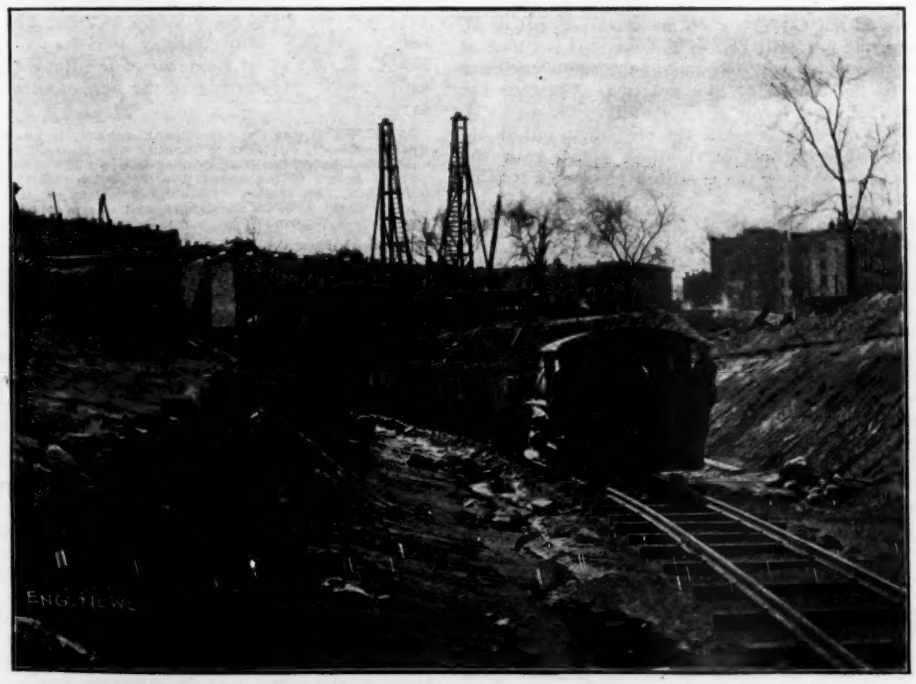


FIG. 109. VIEW OF SPOIL TRAIN EMERGING FROM OPEN CUT SUBWAY WORK.

## EXAMINATION OF A STEEL-FRAME BUILDING FOR CORROSION.

In connection with the discussion concerning the life of steel or ironwork in steel frame buildings and its liability to corrosion, we find in the "American Architect" of Sept. 12, a contribution from an architectural firm in Providence, R. I., which we reprint below. It will be seen that in the case reported there was a gratifying absence of corrosion in every part examined. The letter follows:

As a contribution to the discussion of the effect of cinder concrete in fireproof skeleton construction, the examination which we have been able to make of a building erected five years ago under our supervision, may be of interest.

The building is five stories high, of skeleton construction of "Z-bar" columns and steel beams, which had a coat of red lead before delivery and another coat after erection, with outside walls of brick surrounding the columns in which they are imbedded. The interior steel-columns and the lower flanges of the beams are protected with hard plaster on wire lathing. The floors are of cinder and Portland cement concrete laid on ⅛-in. expanded metal, 3 x 6-in. mesh, 1 in. of the concrete above the expanded metal and 3 ins. below it, and the concrete is carried down to the bottom flange of the beam in the usual manner. The beams in the floor are 4 ft. 6 ins. on centers and were made of a depth to carry safely a live load of 250 lbs. per ft.

The work was, much of it, done in the late fall and early in the winter of 1898, some of it being exposed to very cold weather and most of it to severe rain storms

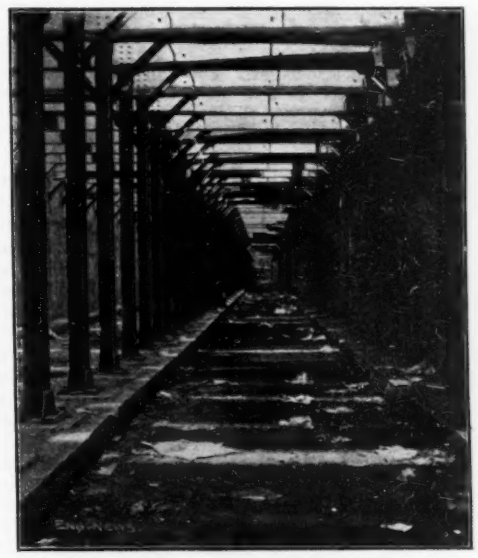


Fig. 110. View Inside Partly Completed Open Cut Subway.

before the building was roofed in. After the building was roofed in and the concrete had thoroughly set, but before the building was finished, a test was made of the strength of the concrete floor-plates, which were loaded up to 900 lbs. per sq. ft. without injury and with but slight deflection.

Since the completion of the building the floors have been almost constantly loaded up to 400 lbs. per ft. in nearly every part of the building.

We have been called upon to drop the first-story floor 2 ft. 8½ ins. while the tenants, wholesale and retail grocers, were carrying on their business, and as the floor which was lowered was used for putting up goods for delivery and also for the delivery of the goods to the teams, the business could not be interrupted, but had to be carried on without interruption while the work was in process.

To do this work it was necessary to expose in very many places the steel frame where it was embedded in the brick wall, where it had been enclosed in the cinder concrete and where it had been protected with steel lath and patent plaster. In no case was there any indication of corrosion, the paint showed scarcely any sign of discoloration and upon scraping the paint the metal under the paint was as bright as when it left the shop. Guide numbers painted on the beams in the shop could easily be read.

The expanded metal in some places showed very slight signs of rust, but not enough to cause any apprehension, and apparently corrosion had stopped.

There seemed to be no indication of rust around the rivets, which had to be driven out, on the brackets which were removed and used again in their new location, or on the "Z-bar" columns behind the brackets.

The concrete plates which were uncovered showed no signs of fracture, and the concrete floors were so solid and firm that the work of cutting the channel to separate the floors from the side walls of the building was no easy task.

There is an elevator enclosed by a partition made of 4-in. terra cotta blocks, laid up in lime mortar between vertical tees, 1¼ x 1¼ x 3-16-ins., set 18 ins. on centers on both sides of the partition to stiffen the same. This partition was plastered both sides with hard plaster to a thickness of about ¾-in. over the terra cotta and ½-in. over the tees. The tees in this partition show more signs of rust than any other of the work, but these tees were not painted and it is evident that the rust was formed before the plastering dried and has not increased since it dried out.

From a very careful examination of this building, which was built without any special care to prevent rust, except the two coats of red lead referred to, there would seem to be no warrant for the apprehension which has been expressed as to the imminent or even remote danger of the destruction and collapse of skeleton buildings with cinder-concrete and expanded-metal floors and beam covering, and plaster column protection.

Very truly yours,
Stone, Carpenter & Willson, Architects.
Providence, R. I., Sept. 1, 1903.

# ENGINEERING NEWS

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As predicted in our issue of Sept. 17, the Hay-Herran treaty with Colombia for the construction of the Panama Canal became void on Sept. 22 through the expiration of the time set for its ratification and the way is now open for the President to make treaties with Nicaragua and Costa Rica and proceed with the construction of the Nicaragua Canal. A Washington dispatch to the New York "Journal of Commerce" under date of Sept. 23, however, states that this will not be done. Alluding to the demands that the Nicaragua route should be taken up, it says:

> The President will not be influenced by these demands. He and Secretary Hay are convinced that the Panama route is so far superior to that by the way of Nicaragua, and that it can be completed in so much less time, that they will be fully justified in making at least one more attempt to reach an agreement with Colombia before taking up the inferior project.

This may be only a newspaper guess at the position of the President and wide of the mark; but if it is true, it is greatly to be regretted. The President, we must believe, would be guided by the judgment of the Isthmian Canal Commission on the relative merits of the two routes, and that body originally recommended the Nicaragua route rather than pay what they deemed an excessive price for the French concession. As for the statement that the Panama Canal "could be completed in so much less time," the Commission's estimate of the time necessary for construction was 10 years for Panama and 8 years for Nicaragua.

Undoubtedly, the President can construe the "reasonable time" set by the Spooner act as allowing him to renew negotiations with Colombia looking to a new treaty; but any such treaty must be ratified by the Senates of both countries, and every sign indicates that the U. S. Senate will approve no new treaty with Colombia, at least until some attempt has been made to secure a favorable treaty with Nicaragua and Costa Rica. As for action by Colombia, that country is about to elect a new President and new congress, and even if it gets through the task without a revolution, it could not take definite action upon any new treaty for nearly a year at best. If such delays constitute a "reasonable time," White House opinions on the strenuous life must have altered of late.

---

The paper on "commercial wrought-iron and steel pipe," which we print in this issue will be read with interest by engineers who have been having trouble with merchant pipe which they have ascribed, in many cases at least, to the change of material from wrought-iron to steel. The author of the paper, it will be noted, strongly defends steel pipe, and maintains that the common idea that it will corrode more rapidly than wrought-iron pipe is unfounded. Whether this be true or not, the fact has to be faced that steel pipe is here, and doubtless here to stay. Actual wrought-iron pipe, made directly from the product of the puddling furnace, is doubtless hardly to be procured in the merchant pipe market now. Such puddled iron as is worked into skelp and pipe finds a more profitable market as boiler tubes.

Meanwhile it is fair to say that the paper in this issue represents the views of the seller of pipe. Some aspects of the pipe question from the user's standpoint are also presented among our "Letters to the Editor" this week, and we shall be glad to hear from any other of our readers who may have experienced difficulties with merchant pipe as now manufactured.

In this connection one point occurs to us on which additional light would be desirable from the user's standpoint. It will be noted that Mr. Riffle urges engineers not to state the quality of material in their pipe specifications, but leave that to the manufacturer, pointing out that it is useless to specify a material which cannot be welded. This means, we presume, that steel skelp for making pipe has to carry a fairly high percentage of phosphorus in order to weld readily. It would be interesting to the user to know just how far this fairly high percentage is carried and about what limit if any is placed on it by the manufacturer. Again, since, in most other lines of steel production, high phosphorus material will not be accepted by the user, is it not a fact that the skelp mill forms a convenient means for disposing of material originally intended for other uses and condemned for too high phosphorus?

We do not say that such material may not be all right and perfectly proper for use; but the user of merchant pipe may feel some pardonable curiosity as to the details of this practice. Whether the remelted steel scrap is an equally good material, whether it ever becomes the custom to put in old scrap of uncertain composition along with the new, and whether the resulting pipe is as reliable under all conditions of service as it ought to be. Of course to the ordinary purchaser of small pipe sizes, for use where a burst or a defective spot is no very serious matter, the quality of the metal does not make much difference and it is for the interest of all parties concerned that the manufacturer should work up material which he can furnish at the lowest cost. When we take such work as steam piping, however, the case is entirely different. The user is entirely justified in wanting the best and most reliable material that can be procured; and engineers have a right to know whether the piping they are erecting to stand the heavy strains of contraction and expansion and possible water hammer is made of a material that would be rejected for boiler plate.

---

Whether the office of City Engineer should be an appointive or an elective one is a question on which opinions differ; but practice in recent years has tended toward making it appointive on the ground that an officer elected by popular vote must first be nominated by a political convention and such a body is more apt to select a man because of his political strength than because of his professional ability.

There is, however, some popular sentiment in favor of the election of officers by the people direct, and this has resulted in some cases in changing the status of the City Engineer from an appointive to an elective official. Such a change has recently been made at Binghamton, N. Y., and we find in the "Republican" of that city the following excellent advice to the voters:

> One of the duties of voters at the caucuses to-morrow will be to choose delegates to the city convention, and the city convention will nominate a candidate for City Engineer. It is the first time the people ever voted for City Engineer, but everybody is aware that City Engineer will be the most important office on the ticket.
>
> It is needless to say that he should be a man of honesty and accuracy. He should be above conniving with rings to increase taxes and pile loads of bonded indebtedness upon the city. It may happen that there will be men on the make in the Common Council. Such things have happened in other cities. If the City Engineer is also on the make, it is likely to go hard with the taxpayers.
>
> The candidate for City Engineer should be what Caesar said his wife should be—"above suspicion." He should be accurate, and when he makes a grade for a sidewalk the property owners interested should be able to depend upon it.
>
> The caucuses to-morrow should select delegates to nominate a model man for City Engineer.

Such public recognition and emphasis of the importance of the City Engineer's work is exceedingly gratifying.

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In the portion of the New York Rapid Transit Railway work which is illustrated and described in this issue mention is first made of the new reinforced concrete construction which the engineers of the Commission have substituted for the steel bent and jack arch construction in a considerable portion of the new subway under way and contemplated. The details of the new construction are given elsewhere and need not be repeated. It will be noticed, however, that the modification is a decided one and is the nearest approach to a purely concrete-steel underground railway structure which has been actually carried out. A somewhat similar use of reinforced concrete is contemplated for the new subway line, work on which has been started, in Philadelphia, Pa., and isolated use of reinforcing rods has been made in the Boston subway work and elsewhere in the rapid transit work in New York. In none of these, except possibly the Philadelphia subway, the plans for which have not been finally settled on and published, is concrete steel used in the same manner or to the same extent as in the New York Rapid Transit construction. Some of the reasons for this change here from the old steel bent and arch-fill construction have been given in the article published elsewhere, but since the work has developed experience with the new construction other reasons of even greater force have come to light. In his last report Mr. Wm. Barclay Parsons, M. Am. Soc. C. E., the Chief Engineer of the Rapid Transit Railway, in referring to the new construction says:

> So successful was the experiment, not only in quality of work produced, but also in rapidity and lessened cost of execution, that it has been decided to apply the same principle of design to the Lenox Ave. extension from 142d to 148th Sts., the station and subway extension at 149th St. and Third Ave. in The Bronx, and to portions of Contract No. 2 in Brooklyn.

To be a little more specific, the new construction shows roughly a saving of about 15 per cent. in materials and of about 20 per cent. in labor, as compared with the other style of structure. To bring out the best results from the new construction, however, it has to be built continuously and where the work enjoys freedom from the necessity of supporting overhead street structures during construction. When the last mentioned necessity exists the steel bent and arch construction possesses important advantages. The bents may readily be erected a few at a time in isolated openings, and as soon as erected and before the arches are filled in they can be used to carry blocking to support street pipes and conduits. It is this property of the old construction which has led to its use in connection with the newer reinforced concrete design in the work planned for future prosecution. The latter is, however, a most interesting departure and possesses merits which make it a strong claimant for the consideration of engineers who have underground railway work to design.

---

One of our contemporaries which is engaged in a violent crusade against the use of concrete as a fireproofing material recently published a report

that a concrete building in Chicago had been condemned and was being taken down. The sort of warfare against concrete which our contemporary is waging may be best understood from the following quotation:

On the Chicago south side there is now in course of demolition that city's first stagger at concrete as a building material.

At the time the building was put up the then acting commissioner, Deputy O'Shea, protested against the permit, assigning the dangerous character of the construction and the method of application and variety of materials as his official reasons.

Notwithstanding these objections, the man with the "drag" put it through, and up went the first frizzled frivolity of the species that had been seen in the middle West.

Up it went, this concrete—twenty years ago—the wonder being that it did not come down in going up, as is the practice, apparently, in our more enlightened modern application of that material.

Down it comes—this concrete house—in the year of grace 1903; that is, it is being carefully and skillfully lifted down by brave and daring workmen in order to prevent its falling down of its own volition.

* * * * * * *

The quality of the concrete is, of course, problematical—perhaps even more difficult of precise definition now than the current supply. It is claimed, however, by the owner and builder that while the work was regarded at the time as necessarily experimental, every precaution was taken to render it safe and secure, the best cement and broken stone and sharp sand being employed in the composition.

* * * * * * * *

Local architects in Chicago express the belief that while the condemnation of the "old concrete house" has been for years a foregone conclusion, it raises anew the question of disintegration as affecting so much of the regular run of that material and the impossibility of determining the real quality and condition of the concrete when once in place, unless the process of application is carried out in the most expensive and laborious way, necessarily involving great loss of time in construction.

In order to ascertain what facts lay at the basis of this piece of fine writing we made inquiry of Mr. Geo. Williams, Building Commissioner of Chicago, and have obtained the following information: The building was erected 20 years ago, and was made with 2 × 6-in. wooden studding, spaced 4 ft. apart. The space between and around these studs was filled in for the height of the basement and two stories with a mixture of cinders and air slaked lime, and the outside was plastered with hydraulic cement mortar. This outside coat of plaster cracked, as might have been expected, and admitted rain to the filling behind it which disintegrated in places; and the moisture being held in contact with the wooden studding, it soon rotted and let the floors down.

This is the "concrete building" whose failure our contemporary holds up as an argument against concrete as a building material! It would be superfluous for us to comment on the "frizzled frivolity" of such trade journalism.

The paper on the design of high masonry dams by Mr. Geo. Y. Wisner, which we print in this issue, calls to mind the discussion of the relative merits of straight and of curved masonry dams which was carried on in this journal a dozen years ago. Since that time enough experience has accumulated with respect to the design of dams and their behavior to make the merits of the question clearer; and there is a strong tendency toward the more general use of curved dams, wherever the topography renders the combination of arch action with retaining wall action advantageous.

Another point emphasized by Mr. Wisner is the liability of a gravity dam to fail by sliding on its base, or in some part of its section rather than by opening the joints on its water face, according to the orthodox theory. This matter has been so well impressed on engineers by the notable dam failures of the last half dozen years that at the present time great care is being taken to tie dams to the bed rock on which they are formed.

The use of steel in connection with masonry structures, too, deserves all the emphasis that can be given to it. Accumulated experience shows that masonry as it is actually laid is far from being a monolithic mass, and when it fails under stress it fails by disintegration. The extensive use of concrete-steel has familiarized engineers with the practice of burying steel in masonry structures and with the value of such steel in enabling the structure to withstand tensile and transverse stresses. It is not that the steel enables the section of the masonry to be reduced, although this may often be the case. The advantage lies in the fact that the steel binds the whole mass together and makes it stronger and more durable than it could be made by any mere increase of its section. It seems reasonable to suppose that the use of embedded steel in masonry will be of the greatest value in tropical countries where temperature strains are most destructive.

American engineering societies and engineering journals are so busy keeping pace with present-day progress that they seldom find time to take a long look backward. On the other side of the water it appears to be different. In the last batch of papers issued by the Institution of Civil Engineers we find one entitled "The Erection of the Nairne Viaducts near Adelaide, South Australia." On examining the paper we find that the viaducts in question were erected 19 years ago, and moreover that they were built by the Edge Moor Iron Co. of Philadelphia. Yet we doubt not there are many young engineers who share the prevalent impression that American bridges and locomotives have never found a foothold in foreign countries until within the last few years.

It should interest those who are thus ignorant of the earlier victories of American engineering to read the following paragraph from the paper in question:

At the Garden Palace Exhibition, opened in Sydney, N. S. W., towards the end of 1879, one of the most interesting engineering exhibits was one span of a road-bridge to be erected over the Shoalhaven River by the Edgemoor Iron Company of Philadelphia, U. S. A. Taking advantage of the presence of Mr. Onward Bates, the Company's Engineer, in Australia, Mr. H. C. Mais, M. Inst. C. E., then Engineer-in-Chief of South Australia, invited him to submit a design for the viaducts. After some slight alterations made by Mr. Mais, the design and strain-sheets were submitted and agreed upon, and a contract was entered into for the supply and erection of the ironwork. The whole of the ironwork and erecting-gear was on board ship in New York Harbor within 30 days of the receipt of the plans and specifications at the Company's office in Philadelphia.

So it seems they knew how to hustle in those days, too!

Further on we are told that the contract price for the supply and erection of the ironwork was £40 per ton for a little over 250 tons. When it is remembered that materials cost more in those days, that the order was a small one, and that the risks of transportation half way round the globe and of final erection had to be assumed, the price seems very reasonable.

The author concludes his paper with the following admirable testimonial to the excellence of American bridge engineering:

When the ironwork was finished everything was found to be absolutely correct. The present Engineer-in-Chief, Mr. A. B. Moncrieff, M. Inst. C. E., recently reported that the viaducts have been used for public traffic for 19 years, and are in first-class condition and perfectly safe.

## A NEW WATER SUPPLY FOR NEW YORK CITY.

The need of a wider margin between the water consumption and the available water supply of New York city has long been realized, and various investigations have been made within the past few years to determine the best sources for such a supply. The Ramapo scheme which roused such a stand of public opposition in 1899, contemplated a supply to be furnished by a private company at a high rate per million gallons, and this was followed in 1900 by the Merchants' Association investigation and also an independent investigation made by Mr. John R. Freeman for the then City Controller. Each of these investigations showed that ample water supplies could be obtained at less cost to the city than that offered by the Ramapo Co.

Following these two reports, the Water Department of New York city began surveys in the Catskill region, and all these projects and investigations had been preceded some years earlier, prior the formation of Greater New York, by extensive investigations on behalf of the old city of Brooklyn.

In December, 1902, the present New York municipal administration wisely concluded to institute a comprehensive and authoritative investigation of the water supply of the city, including the alleged waste of water from the existing distribution system, and the future needs of the city as to quality, quantity and pressure. Accordingly a commission was appointed, consisting of Prof. Wm. H. Burr, Chairman, and Messrs. Rudolph Hering and John R. Freeman. The commission was authorized to make a thorough inquiry, was given nearly a year for its work, and was liberally provided with funds. Each of the members of the commission appointed was already more or less familiar with the ground to be covered. Mr. Freeman in particular had made the special report already mentioned, and Mr. Hering was a member of the committee which prepared the Merchants' Association report.

The commission speedily gathered a staff of engineers who had been connected with some of the largest water supply works in this country, besides which it had the active co-operation of the New York Department of Water Supply, Gas and Electricity, under the direction of Messrs. Robert Grier Monroe, Commissioner, and Nicholas S. Hill, Jr., Chief Engineer.

The first progress report of this commission, setting forth its main conclusions, is printed in full elsewhere in this issue.

Briefly summarized, the commission recommends the provision of an additional supply of 500,000,000 gallons per day of water, drawn from the upland reaches of tributaries of the Hudson lying north of the present Croton drainage area. This would be a high level, gravity supply of filtered water, brought to the city through one large aqueduct, mostly in tunnel. An aqueduct of 500,000,000 gallons daily capacity for practically its whole length, and dams, reservoirs and filters to provide a supply of 200,000,000 gallons daily, would cost about $50,000,000, and such works are recommended for immediate construction.

Rarely is a report made on a great engineering project under conditions so favorable to the universal endorsement of men, methods, and conclusions.

The report, as already indicated, follows a number of years agitation for a new supply, during which several important studies of the subject were made. The present investigation may be considered largely as a continuation, under more favorable circumstances, of the former ones, and since it has been so largely under the direction of the same guiding minds its conclusions are naturally in general accord with those of the earlier reports, modified, however, by later developments, more thorough research, and particularly by the fact that this has been an official and designedly a final investigation.

Beyond all these weighty reasons for confidence in the report we have the inherent reasonableness of its conclusions. It has long been evident that an additional water supply for New York city must be taken either from the Hudson River above Poughkeepsie, which must be first filtered and then pumped to the city, or else from some upland stream or streams whence it could be delivered by gravity.

The engineers on the Merchants' Association committee in 1900 recommended the former source. Mr. Freeman advised the latter. Fortunately for the city of New York the further and more detailed studies of the present Commission show that an upland gravity supply can be delivered near the northern limits of the city, at a comparatively high elevation (some 300 ft.) for about the same unit cost as a pumped and filtered supply from the Hudson.

If the choice lay between the filtered Hudson and the unfiltered upland water probably not a few of the most advanced engineers and sanitarians of the day would prefer the filtered river water, but happily the plans and estimates include filtration for the upland water as well. In addition, the latter will have the great benefit of large storage, with sedimentation and bleaching of the water, and with the marked reduction, if not elimination, of such pathogenic germs as may find their way into water gathered from even the most sparsely populated drainage areas. In short, the commission's plan combines the best features of the two main alternative plans.

Besides this, in recommending that the entire water supply of New York city, the present as well as that proposed to be furnished, shall be filtered, the Commission proposes to put New York city in line with the most advanced ideas of sanitarians in this country and with the best practice of the water-works engineers of Europe.

This report illustrates to a remarkable degree the great value of high class of expert advice in modern municipal administration. An excellent

illustration of this is the work which was done by the Commission's specialist on pumping engines. We learn that the tests made by him on the high service pumping engines in Manhattan Borough showed one pump with 60% and another with 65% of slip. Overhauling these and other pumps resulted in so great an increase in capacity as to permit shutting down one or more engines. Altogether, a saving of thousands of dollars a year to the city was thus effected. Indications are that at the Brooklyn pumping stations, where much more water is handled, $75,000 a year can be saved to the city by proper attention to the pumping machinery.

Less obvious in importance, so far as the general public is concerned, but perhaps even more striking to engineers, are the prominence of the bacterial, chemical and physical studies of water as a part of the investigation. The detailed results of these are not given in the report, but their importance is plainly evident, as is also the promising special work on the character and amount of the underground water supplies of Long Island. These waters have long been notable and the results of the special joint study by the commission and the U. S. Geological Survey will be awaited with great interest.

Still another phase of specialization worthy of mention is the water waste investigation by means of the pitometer, in connection with plumbing and sewer inspection. The most recent developments in this line were set forth at length in our issue of Sept. 24.

While large credit is to be awarded to the specialists who have carried out these various lines of work, to give such credit detracts nothing from the value and importance of the work done by the members of the Commission themselves. It has been their task to bring all the work of the specialists together, submit the results to analysis, and base their final conclusions upon the largest possible knowledge of the problem. Such work demands the highest grade of professional talent. The results attained in this case thoroughly justify the policy of making up such commissions entirely of engineers instead of the opposite practice, still too common, of appointing a set of more or less distinguished citizens, or politicians, to act either as amiable figure-heads or else to interfere with that little knowledge which is proverbially dangerous.

## LETTERS TO THE EDITOR.

### Making Sun-Prints from a Print.

Sir: If "Young Engineer," who inquired in your issue of Sept. 10 how to make a sun-print from another print, will use thin brown print paper instead of blue he will get what he wants. Several firms make such paper. The light goes through the white lines and he will secure a nearly white ground on the print with dark lines. If he still wishes to use blue prints he can make the white lines more transparent by carefully going over them (as in tracing) with the following varnish: Canada balsam, 1 part; spirits of turpentine, 3 parts; alcohol, 1 part, all by volume. Use the varnish as you would ink. It is a nuisance and thin brown prints make better negatives.

Engineer.

Chicago, Ill., Sept. 20, 1903.

### Waterproofing Masonry.

Sir: Referring to the request of Capt. H. M. Chittenden and your attached editorial of Sept. 10, concerning the protection or waterproofing of exposed surfaces of stone or brick masonry, kindly grant me space for a few comments on the subject which, while not based on actual experiment, may perhaps lead to further investigation and throw more light on this important question.

Your remarks on the paraffine getting into the pores of the stone of the Egyptian obelisk, call to mind the method suggested by some author of filling up the pores with some insoluble substance which would thus prevent the moisture from getting into the stone.

Calcium silicate is a common ingredient of most stones and an insoluble silicate of calcium or lime can be obtained by treating the stone with a weak solution of silicate of potash and then applying a solution of chloride of calcium. Calcium silicate is more durable than sandstone, and a solution of this would not discolor a stone, since both solutions of which it is made are colorless, and the resulting insoluble compound is white. Neither of the chemicals used is very costly.

In the case of a monument or masonry already built, these solutions can be applied as a wash, but in the case of new masonry, the stone should be soaked.

Very respectfully, Geo. F. Syme.

Raleigh, N. C., Sept. 21, 1903.

### The Comparative Strength of East River Bridges; Existing and Proposed.

Sir: In a communication to the Engineering News, published in your issue of July 30, and in a letter to the Board of Aldermen, dated July 31, Bridge Commissioner Lindenthal asserts that the proposed Manhattan bridge, if built on the new plan, originated by himself, would have "a capacity 40% greater for the cables and 80% greater for the stiffening features, than provided in the first plan, as well as that of Williamsburgh bridge."

This claim is based on a comparison between the live loads assumed for the different bridges. It was stated that the Williamsburgh bridge, as well as the first design for the Manhattan bridge, was calculated for a live load of 5,700 lbs. per lin. ft. for the cables, and for one of 4,500 lbs. per lin. ft. for the stiffening trusses, while the new design was alleged to be proportioned throughout for a live load of 8,000 lbs. per lin. ft., and, in addition thereto, that it was capable of carrying safely an emergency load of 16,000 lbs. per lin. ft.

It was specially pointed out to the Aldermen that "no such emergency load (double the working load) was considered or provided for in the Williamsburgh bridge, or in the first plan for the Manhattan bridge."

There is something wrong in these statements. They might be correct if the strength of a structure could be gaged by a mere assumption of live loads on paper without considering the factor of safety of the bridge, but actually the strength of these bridges depends on the stresses the loads produce in the cables and their relation to the strength and elastic limit of the latter.

If Mr. Lindenthal had calculated the tensions in the cables of the Williamsburgh bridge and of the first design for the Manhattan bridge, as well as those in the eye-bar chain of his design, and compared these stresses with the respective strength of the cables, he would have been forced to reverse his statement and confess that, instead of being 40% stronger, the eye-bar design is at least 55% weaker than the old design or than the Williamsburgh bridge. All data necessary for making these calculations are contained in Mr. Lindenthal's different publications, and the formulas for determining the minimum and maximum stresses in suspension cables are so simple and well known that they need not be repeated here.

I have carried out these calculations for the old and new design of the Manhattan bridge, for the Williamsburgh bridge, and for the old Brooklyn bridge, and I submit herewith the results, which, for convenient study, have been placed in tabular form.

It will be noticed that two values are given for the strength of the eye-bar chain—one based on a tenacity of 42.5 tons per sq. in. with an elastic limit of 24 tons, as found in testpieces according to the report of the Board of Experts, and another value based on the "probable" strength of full-sized eye-bars which, according to past experience, is 22% less than that of testpieces, with a reduction of about 10% in the elastic limit. The true strength of this chain will only be known after a sufficient number of eye-bars, of the required huge dimensions, have been manufactured and tested. It is different with the strength and elastic limit of wire cables, where no guessing is necessary. The values given in the table are the results of two tests of every wire, aggregating over 60,000, in the cables of the Williamsburgh and Brooklyn bridges. The figures in column 7 are usually called the "factors of safety" for the bridge, which means that the strength of the cables is three, four (or whatever the factor may be) times stronger than the stresses in the cables, due to the combined dead and live loads of the bridge. These factors show, for instance, that for a live load of 8,000 lbs. per lin. ft. (equal to 57 lbs. per sq. ft. of floor), the factor of safety of the Williamsburgh bridge is from 47 to 88% greater than that of the eye-bar design for the Manhattan bridge, The difference in the percentage refers to the strength of the chain, whether it be that of testpieces or the more probable strength of finished eye-bars. In place of being 40% stronger, as claimed by Mr. Lindenthal, the eye-bar design is, therefore, at least 40% weaker than either the Williamsburgh bridge, or the original design for the Manhattan bridge! Even the factor of safety of the old Brooklyn bridge is, for the same live load, from 16 to 50% greater than that of the Manhattan eye-bar bridge.

Many engineers have become accustomed to refer the degree of safety of a structure, not to the ultimate strength of the material but to its elastic limit. There are good reasons for recommending this practice, though it has never come into general use. All theoretical calculations are only correct within the elastic limit of the material, and it is universally agreed by all engineers that any bridge or part of a bridge, strained beyond its elastic limit, has become crippled and must be considered unsafe. In column 8 of the table the total elastic limit of the wire cables and eye-bar chains are recorded, and the ratio of the maximum stresses to the elastic limit of the cables are enumerated in column 9. According to the latter figures, "factors of safety for elastic limit," the comparative safety of the eye-bar bridge will appear in a much more unfavorable light than if referred to the ultimate strength. The safety of the Williamsburgh bridge will, for the same live load, be from 120 to 145%, and that of the old Brooklyn bridge from 78 to 100% greater than the safety of the Manhattan bridge after the new design.

To comply with the popular understanding of a safety factor, I have added two more columns to the table, which give the actual carrying capacity of each bridge, after the elimination of its own weight. In column 10 factors are given with which each assumed live load must be multiplied to produce, in combination with the dead load, stresses in the cables equal to their elastic limit. In column 11 these multiplications are carried out and the exact amount of live load per square foot of floor is registered, which each bridge is capable of sustaining before it is crippled by exceeding the elastic limit of the cables.

Comparing the latter figures, it will be seen that the Manhattan bridge, as originally designed, is, comparatively, the strongest of all East River bridges; that, for the same degree of safety, the carrying capacity of the Williamsburgh bridge is from 2 to 2½ times, and that of the old Brooklyn bridge from 1½ to 2 times greater than the carrying capacity of the Manhattan bridge, as at present proposed to be constructed!

Therefore, the boasted strength of the eye-bar bridge is turned into weakness, and the "emergency load" is merely a name or an illusion, but no practical fact for proving therewith, the merits of the proposed Manhattan bridge!

The facts, elucidated in the above-described table, are alone sufficient to condemn the new design for the Manhattan bridge, even if it had no other objectionable features. One of these latter is the increased cost of the bridge. It stands to reason, without going into calculations, that the eye-bar design, which, in its present form has a superstructure twice as heavy as that of the original design, must cost more than the latter. Mr. Lindenthal, in his letter to the Aldermen, acknowledges that his bridge would cost "only" 20% more than the Williamsburgh bridge. I venture to predict, however, that the Manhattan bridge will not be built on the present plan. After the true facts, as explained herein, are once known and understood, the public will never permit that a new bridge be erected which has less safety than the old Brooklyn bridge. To build an eye-bar chain bridge of the same strength as the Williamsburgh bridge means that the chain must be at least five times the size designed in the present plan, which will increase the cost of the bridge enormously. I have repeatedly proved by figures that the Manhattan bridge, built with eye-bar chains and constructed on the more economical plan of the Williamsburgh bridge, will cost at least $3,000,000 more than the latter, but if built on the Lindenthal plan it will cost $5,000,000 more. The correctness of the calculations leading to these results, which were published months ago, has never been disproved.

Another feature of the new design for the Manhattan bridge, on which Mr. Lindenthal has laid stress is its alleged stiffness. Mr. Lindenthal states that the center span, if loaded with a live load of 16,000 lbs. per lin. ft., while the end spans are unloaded, would deflect only 23 ins., that "the suspension bridge would be as rigid as a truss bridge," and that "any accumulations of loaded cars would affect its strength as little as if it were on solid ground." I have reasons to doubt the correctness of these statements, but if they were true they would only prove that Mr. Lindenthal's plan does not embody the genuine suspension bridge principle, and that it was designed for conditions not characterizing a long-span suspension bridge, but a short-span truss bridge! There is no more sense in constructing a suspension bridge as rigid as a truss bridge than there is in attempting to make a steel truss as rigid as a stone bridge. Each type of bridge has its peculiarities. If a stone bridge were as flexible as a truss bridge it would collapse, and a truss bridge would collapse if it were as flexible as a properly constructed suspension bridge. The advantage of a suspension bridge is its flexibility combined with safety and usefulness, which makes it possible to span a great waterway at a much more reasonable cost than can be done with any other kind of bridge. An attempt to impose on one type of bridge the conditions necessary for another type, especially if, as in the case of the Manhattan bridge, it is done at an enormous cost and at the expense of the strength and safety of the structure, is, in my opinion, a serious blunder.

At another opportunity I have stated that the stiffening construction of a suspension bridge is a luxury, and not a necessity. It can be dispensed with, without impairing the strength of the bridge, but for the comfort of passengers and teamsters, it is desirable to stiffen the floor sufficiently to prevent marked and abrupt changes in the grade, and strong vibrations. All rigidity, beyond making the bridge practical and comfortable, is not only unnecessary and wasteful, but also undesirable. The stiffening trusses of the Brooklyn bridge weigh less than 2,000 lbs. per lin. ft., but is there anybody who was ever prevented from crossing that bridge or who felt uncomfortable on account of the flexibility of the floor? On the contrary, in the center of the bridge where the motions are greatest they are noticed much less than within the towers where the trusses may be said to rest on solid ground, but where a passing heavy vehicle causes short and sharp vibrations

which are distinctly felt and are much more disagreeable than the easy and long undulations of the center span.

A similar observation can be made on the Cincinnati bridge. The end spans have the stiffness of a truss bridge, but the motions of the floor, though smaller than those of the center span, are felt much more distinctly than on the river span, which has only the stiffness of a suspension bridge. The stiffening trusses of the Williamsburgh bridge weigh 3,800 lbs. per lin. ft. That this bridge is very rigid was proved by the traveling crane for erecting the superstructure which weighed 80 tons, distributed over a space of about 25 to 30 ft., and which, before the trusses were erected, caused so little local deflection in the floor that an unpracticed eye did not notice it. The stiffening construction of Mr. Lindenthal's eye-bar bridge weighs 5,250 lbs. per lin. ft., or 38% more than that of the Williamsburgh bridge. The late Geo. S. Morison, one of the five experts appointed for examining the new plan for the Manhattan bridge, told me personally that he considered the Williamsburgh bridge much too rigid, and that the heavy stiffening trusses were unnecessary and wasteful. According to his judgment, a flexible truss not over 10 ft. high would be ample for stiffening a highway bridge spanning the East River. I am not sure whether I would endorse all of Mr. Morison's suggestions about stiffening trusses, but I fully agree with his general idea that it is not unbounded rigidity which an economically and scientifically constructed suspension bridge should possess, but a reasonable degree of flexibility!

W. Hildenbrand, C. E.

1 Broadway, New York, Sept. 2, 1903.

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**(A proof of the above letter was submitted to the Commissioner of Bridges with an invitation to present a reply for publication in the same issue as Mr. Hildebrand's letter. We append the Commissioner's response.—Ed.)**

Sir: I appreciate your courtesy in affording me the opportunity to reply to Mr. Hildenbrand's communication. A wide discussion of such an important structure as the proposed Manhattan bridge can only be welcomed. Had such discussion and an examination of plans by a board of experts preceded the construction of the Brooklyn, or, later, of the Williamsburg suspension bridge, an improvement in design would probably have resulted in both cases.

First I desire to refer to the repeated statement by Mr. Hildenbrand, that the Manhattan bridge with eye-bar chains will cost from three to five million dollars more than with steel wire cables. Mr. Hildenbrand asserts that he has proved this in an article published in Engineering News, March 12, 1903, and that his estimates given therein have never been disproved. Mr. Hildenbrand had an opportunity, at his request, to present his so-called proofs to the board of five bridge experts appointed by Mayor Low to examine the design for the Manhattan bridge. The board gave him a patient hearing, but after careful consideration of his statements, concluded to take no further notice of them. Neither did I consider it necessary to reply to them. The "proofs" were mostly based on imaginary standards and conditions, as to quality of steel, degree of stiffening, factors of safety, etc. They had no relation to the actual design and strain sheets of the Manhattan bridge, which Mr. Hildenbrand had never seen. He alleged, for instance, that the eye-bar chain would weigh 46,440,758 lbs. for the working live load of 5,700 lbs. per lin. ft., assumed by him, the same as that of the cable in the Williamsburg bridge. But the actual design provides for a working load of 8,000 lbs., which is 40% greater, and yet with that greater live load the chains will weigh only 29,040,000 lbs., as shown in my table of comparison of eye-bar chains and steel wire cables (published in Engineering News, June 25, 1903).

Mr. Hildenbrand also asserted that no nickel steel eye-bars of the desired quality and size could be obtained. Further, he charged that the Bridge Department wasted the city's money.

The entire steel superstructure, including chains, towers, stiffening, floor, etc., of the Manhattan chain bridge (designed for 8,000 lbs. working load and 16,000 lbs. emergency load) will not exceed 90,000,000 lbs., and when estimated at current prices would cost about $5,600,000.

If a steel wire cable structure would cost $5,000,000 less, as Mr. Hildenbrand asserts, then the entire superstructure for a wire cable bridge, including towers, should cost the city only $600,000, or about $200 per lin. ft. The superstructure of the much lighter Williamsburg bridge cost $1,420 per lin. ft. That should suffice to show the absurdity of Mr. Hildenbrand's assertion.

Equally preposterous and misleading is Mr. Hildenbrand's latest communication and table, purporting to be a comparison of the suspension bridges over the East River, existing or designed on the basis of their cables alone. His data for the original design for Manhattan bridge, the plans for which were never completed, are not authentic.

According to that table, the Brooklyn bridge, for instance, could sustain a live load of 16,000 lbs. per lin. ft., with a factor of safety in the cables of 1.815. Yet long before that load were imposed upon one-half of the bridge, the stiffening girders and flooring would be wrecked. The full load would upset the anchorages, and the cables, torn from their roots, would knock the stone towers over into the East River.

What is the sense of comparing suspension bridges of radically different design on the basis of strength of cables alone, and ignoring the strength of other parts of the bridge? The cables may have a factor of safety of ten, and yet if the floor and stiffening system are weak the bridge would be unfit for use.

Would any one think, in comparing arch bridges (such, for example, as those shown in Figs. 1 and 2, which represent the inverted forms of the Manhattan and Williamsburg suspension bridges), of basing their carrying capacity upon the cross section of the arch ribs, without also taking into consideration the weight and strength of the stiffening or bracing of the arches and that of the floor?

It is one of the first rules of good bridge designing that a bridge and every member in it must be safe under any condition or grouping of live loads which will produce the maximum stress. That applies to suspension bridges just as well as to arch or other bridges. The only difference is that a suspension bridge, being in stable equilibrium, cannot collapse if the stiffening system is weak. Upon that is based the greater safety of the suspension bridge in case of accidents, as, for instance, when a train derailment might knock out one or more braces. An erect arch would be destroyed, but the suspended arch would still stand. Depending upon that feature, many suspension bridges have been built with a weak stiffening system or none at all. It is not a permissible economy to do this, however, in a suspension bridge intended for a heavy railroad traffic (elevated and trolley) in a city like New York.

Mr. Hildenbrand's table gives factors of safety for the cables when strained to the maximum from live load, that is, when the span is fully covered.

No stiffening truss is required for that condition of loading. He could have safely left off the weight of the

TABULAR COMPARISON OF THE STRENGTH OF EAST RIVER BRIDGES EXISTING AND DESIGNED.

| 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 |
|---|---|---|---|---|---|---|---|---|---|---|
| Name of Bridge. | Dead load per lin. ft. | Live load per lin. ft. | Maximum tension in wire Cables and average tension in eye-bar chain, due to combined live and dead loads. | Cross section of cables or eye-bar chains. | Ultimate strength of cables or chains. | Ratio of stresses enumerated in column 4 to ultimate strength of cables or chains. | Elastic limit of cables or chains. | Ratio of stresses enumerated in column 4 to elastic limit of cables or chains. | Factors with which the assumed live loads must be multiplied to produce stresses in cables or chains equal to their elastic limit. | Total live load per sq. ft. of floor necessary for reaching the elast. limit of the cables and crippling the bridge. |
| | *lbs.* | *lbs.* | *tons.* | *sq. inches.* | *tons.* | | *tons.* | | | *lbs.* |
| "Manhattan" (Original design) (Wire cables.) | | 5,700 (= 48 lbs. per sq. ft.) | 19,112 | | | 4.688 | | 3.976 | 12.072 | |
| Span.....1,465 ft. | 15,500 | 8,000 (= 67 lbs. per sq. ft.) | 21,185 | 800 (uniform.) | 89,600 (= 112 tons per sq. in.) | 4.224 | 76,000 (= 95 tons per sq. in.) | 3.588 | 8.600 | 576 |
| Deflection.....$\frac{1}{9}$ | | | | | | | | | | |
| Width of floor..119 ft. | | 16,000 (= 134 lbs. per sq. ft. of floor.) | 28,395 | | | 3.156 | | 2.676 | 4.300 | |
| "Manhattan" (New design) (Eye-bar chains.) | | 5,700 (= 40 lbs. per sq. ft.) | 29,538 | | (a) 91,630 (acc. to strength of testpieces = 42.5 tons per sq. in.) | (a) 3.102 (b) 2.419 | (c) 51,744 (acc. to elastic limit of testpieces = 24 tons per sq. in.) | (c) 1.752 (d) 1.576 | (c) 6.006 (d) 4.839 | |
| Span ..... 1,470 ft. | 32,250 | 8,000 (= 57 lbs. per sq. ft.) | 31,328 | 2,156 (average.) | (b) 71,471 (acc. to probable strength of finished eye-bars = 33.15 tons per sq. in.) | (a) 2.925 (b) 2.281 | (d) 46,569 (acc. to probable elastic limit of finished eye-bars = 21.6 tons per sq. in ) | (c) 1.652 (d) 1.486 | (c) 4.278 (d) 3.447 | (c) 242 (d) 194 |
| Deflection.... ....$\frac{1}{9}$ | | | | | | | | | | |
| Width of floor..141 ft. | | 16,000 (= 113 lbs. per sq. ft. of floor.) | 37,555 | | | (a) 2.439 (b) 1.903 | | (c) 1.379 (d) 1.240 | (c) 2.139 (d) 1.723 | |
| "Williamsburgh" (Wire cables.) | | 5,700 (= 40 lbs. per sq. ft.) | 21,003 | | | 4.750 | | 4.031 | 12.345 | |
| Span .....1,600 ft. | 15,632 | 8,000 (= 57 lbs. per sq. ft.) | 23,268 | 891.28 (uniform.) | 99,761 (= 111.9 tons per sq. in.) | 4.298 | 84,672 (= 95 tons per sq. in.) | 3.648 | 8.795 | 496 |
| Deflection.....$\frac{1}{9}$ | | | | | | | | | | |
| Width of floor..141 ft. | | 16,000 (= 113 lbs. per sq. ft.) | 31,143 | | | 3.203 | | 2.719 | 4.397 | |
| "Brooklyn" (Wire cables.) | | 2,500 (= 32 lbs. per sq. ft.) | 12,982* | | | 4.025 | | 3.479 | 11,614 | |
| Span .....1,595.5 ft. | 8,204 | 4,454 (= 57 lbs. per sq. ft.) | 15,352 | 590.4 (uniform.) | 52,248 (= 88.5 tons per sq. in.) | 3.404 | 45,164 (= 76.5 tons per sq. inch.) | 2.942 | 6.528 | 370 |
| Deflection .....$\frac{1}{11.5}$ | | | | | | | | | | |
| Width of floor,78.5 ft. | | 8,908 (= 113 lbs. per sq. ft.) | 20,753 | | | 2.518 | | 2.176 | 3.260 | |
| | | 16,000 (= 204 lbs. per sq. ft. of floor.) | 29,345 | | | 1.780 | | 1.539 | 1.815 | |

*NOTE.—The cables of the Brooklyn bridge are assisted by a system of stays, hence the tension in these cables is about 10% smaller than in cables unassisted by stays.

stiffening trusses and obtained still higher factors of safety for the cables. But when that same live load covers only a part of the span, then the stiffening trusses come into action and their weight is a most important part of the dead load borne by the cables. It cannot be omitted or minimized in a fair comparison.

The floor construction and, especially, the stiffening of the wire cable bridges in Mr. Hildenbrand's table, are very inferior to that of the Manhattan chain bridge, with which they are compared. The Manhattan Bridge is designed with a fireproof buckle-plate floor, absent in the other bridges. A comparison of that sort is obviously of no value.

The weight of the stiffening frames depends upon their form and upon the degree of rigidity desired in the bridge. For a heavy railroad traffic, such as all East River bridges must accommodate, the rigidity desired is the same as for an erect arch bridge, or for a cantilever bridge of equal span. Estimates show that a cantilever bridge (for length of span up to 1,800 ft.) is cheaper than a steel wire cable suspension bridge combined with parallel chord stiffening trusses, which is not as rigid. But the suspension type has been chosen for the Manhattan bridge on account of its better appearance. The stiffening frames are attached directly to the four chains which form the upper chords. These frames are equal in rigidity to stiffening trusses in the average 62 ft. high, and weight on the average 5,200 lbs. per lin. ft. of bridge, if constructed of structural steel. But if constructed of nickel steel, their weight will be only 3,910 lbs. per lin. ft. These stiffening frames will make the suspension bridge as rigid as a cantilever bridge for a live load of 8,000 lbs. and an emergency load of 16,000 lbs. per lin. ft.

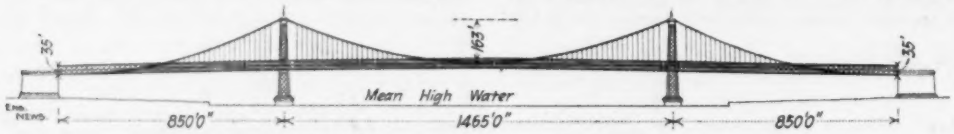



FIG. 1. DESIGN FOR THE MANHATTAN BRIDGE WITH WIRE CABLES.

The same unit stresses which are sanctioned by good practice for railroad cantilever, arch and truss bridges, are applicable to suspension bridges. The factor of safety in the Manhattan chain bridge will relatively not be less than in the great Forth cantilever bridge in Scotland (the largest bridge in existence, having among others two spans of 1,710 ft. each), and over which trains pass at 60 miles per hour.

Stiffening trusses for the same live loads, but with parallel chords 35 ft. high, combined with steel wire cables, would have an average weight of 10,600 lbs. per lin. ft., or about double the weight of the chain stiffening. Notwithstanding their greater weight, they would be less stiff, because of their lower height.

Why should the city build inferior flexible steel wire cable suspension bridges over the East River when they are no cheaper and are less serviceable than rigid cantilever bridges? A chain suspension bridge, with the stiffening system as in the Manhattan bridge, is, however, just as rigid as a cantilever structure and is no more costly. It also has a better appearance than can be given to a large cantilever structure.

In the Williamsburg bridge, the stiffening trusses are 40 ft. high at the towers and 36.75 ft. at the center, and weigh 3,800 lbs. per lin. ft. The trusses are continuous through the towers and intended for a live load of only 4,500 lbs. per lin. ft. For this load the unit stresses are practically the same as in the stiffening frames of Manhattan bridge for the working load of 8,000 lbs. The working load for the Williamsburg bridge should, however, be the same as for the Blackwell's Island cantilever bridge, which has the same number of tracks and same roadway and promenade capacity. This bridge is computed for a working live load of 6,300 lbs. and an emergency load of 12,600 lbs per lin. ft. of bridge. No rules against the congestion of loads, such as are necessary on the Brooklyn bridge, will be needed on the Blackwell's Island cantilever bridge, nor on the chain-stiffened Manhattan bridge. But for the stiffening trusses in the Williamsburg bridge the question has already arisen, whether the chords would not be overstrained for certain positions of congested loads. It may be very rare, but must be considered. The trusses would be safer if, for the same weight, they were of lower height. That would give more flexible trusses with heavier chord sections. Low, flexible trusses are relieved of a larger proportion of the live load through the greater sagging of the cables. Such a structure is inferior in rigidity, but for slow speeds would be stiff enough.

The Brooklyn bridge is an example of an insufficiently stiffened suspended structure, carrying the concentrated load of elevated railroad trains, besides trolley traffic and roadway traffic. It has four tracks, over which trains and cars must move comparatively slowly. Stringent rules are necessary to keep the cars evenly distributed, and to prevent rupture of the stiffening trusses, which are of an antiquated pattern, defying computation. With all possible precautions against overstraining and buckling of the chords, it cannot be prevented altogether. Repairs are continuous and expensive. The rebuilding of the suspended structure cannot be much longer deferred. The cables are safe enough for present loads, but they would not be so if the floor construction and the stiffening trusses were of modern and heavier construction. Nor would the towers and anchorages suffice for the heavier loads. Cables, towers and anchorages can, however, be utilized for a strong double deck structure, when the time comes for its reconstruction. That is a story for another occasion.

The experience with the stiffened railroad suspension bridge (800-ft. span) over the Niagara River (rebuilt six years ago with a rigid arch bridge) proved clearly that it was poor economy to save in the sections and weight of the stiffening trusses. They had a depth of 20 ft. or about 1-40 of the span. They had a flexibility which Mr. Hildenbrand would call reasonable. But trains could move over the bridge only at a man's gait, the deflections were excessive, and the steel chords were repeatedly ruptured. And although the iron wire cables had an ample factor of safety, the bridge had finally to be taken down, and was replaced by a rigid arch bridge on which there is no restriction of speed.

According to Mr. Hildenbrand, the stiffening of the cables is a luxury and not a necessity. As well might he claim that the rails in a railroad track are a luxury, because the older construction of flat bars, spiked down to wooden stringers, carried the old cars safely enough.

Mr. Hildenbrand calls a scientifically stiffened suspension bridge, passed upon by a board of experienced bridge experts, a serious blunder. His preference is for stiffening trusses designed by rule of thumb, as in the old structures, and he refers to the Cincinnati suspension bridge over the Ohio River, which he repaired, as the example of a good suspension bridge.

To bridge engineers the stiffening system in that bridge appears unique, in being probably the most unscientific in existence. In that respect the system in the Brooklyn bridge is its only rival. The trusses are fixed at the towers, but are shallower at that point than at the center, having a curved top chord for beauty's sake. The chords are cut in the center and have slip joints at these points, making the wind bracing illusory. Inclined stays extend from the towers, presumably assisting the cables, but actually having no ascertainable value. The cost of the superstructure is greater than that of a cantilever bridge. That is the nearest that Mr. Hildenbrand came to building a bridge of his own designing, his experience having been mostly in the making of wire cables.

Mr. Hildenbrand quotes the late Mr. Geo. S. Morison as saying that he considered the Williamsburg bridge as much too rigid, and the heavy stiffening trusses wasteful; also that he considered trusses of 10 ft. high enough for a highway bridge over the East River.

I believe I am familiar with Mr. Morison's views, which he had discussed with me. I agreed with him that floor stiffeners of 10 ft. in height were ample for a highway suspension bridge of 1,600 ft. span and with a flat catenary. But none of the East River bridges are purely highway bridges. They are more for railway traffic than for wagon and pedestrian traffic. The rail traffic is the principal traffic, and requires rigid structures.

Mr Morison's view of the Williamsburg bridge trusses, as expressed to me, was that they would have been stronger if made only one-half the height, but with the same weight of metal in them. In that view I also agreed with him, but this is a quite different opinion from the one ascribed to Mr. Morison by Mr. Hildenbrand.

Mr. Morison's opinions of stiffening trusses are embodied in his paper, "A Study in Suspension Bridges" (Trans. Am. Soc. C. E., Volume XXXVI., 1896, p. 360). The views he expressed to me were consistent with those in his paper, and also with the report of the Board of Experts upon the Manhattan bridge, of which he was chairman.

To sum up, Mr. Hildenbrand advocates the use of steel wire cables as against eye-bar chains. It is evident from his letters that he cannot discuss the question unbiased. He sees nothing good in eye-bar chains. The proposed nickel steel cannot be obtained for them. If obtained, the chains would weigh five times as much as the wire cables. That makes a difference of several millions for the chains alone to begin with. Then he protests against using the same factor of safety in suspension bridges which good practice has established for arch, truss, or cantilever bridges of same span and for same load conditions. He wants at least twice that factor of safety for the chains, but will be satisfied if the stiffening trusses, being a mere luxury, have no factor of safety to speak of, in order that the genuine principle of suspension bridges, which is flexibility, as exemplified in the antiquated Brooklyn and Cincinnati suspension bridges, may be fully preserved. He also prophecies that no other kind of bridge will be built over the East River.

It may occur to thoughtful engineers, that if Mr. Hildenbrand's prophecies have the same value as his theories on suspension bridges and factors of safety, or his knowledge of nickel steel, then it is no greater than the value of the weather prophecies in a Farmer's Almanac.

G. Lindenthal, Commissioner of Bridges.

New York City, Sept. 26, 1903.

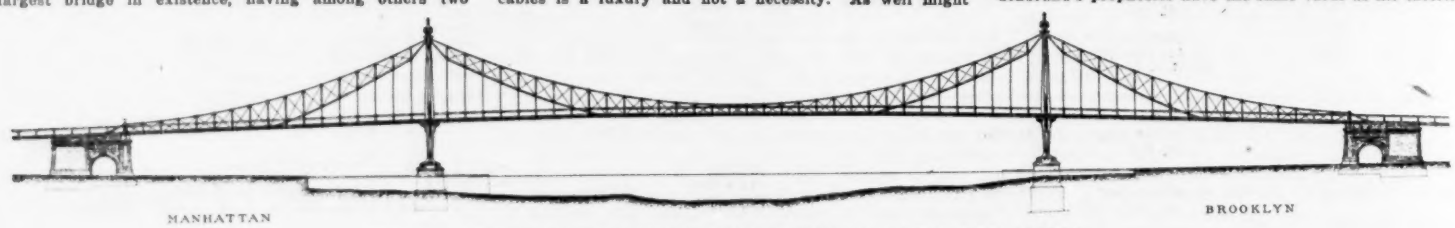



FIG. 2. DESIGN FOR THE MANHATTAN BRIDGE WTH EYE-BAR CABLES.

## Substitution of Under-Weight Steel Pipe for Wrought-Iron Pipe.

Sir: Will you invite some discussion on the present "state of the art" in the manufacture and use of wrought-iron pipe?

When one places an order for *wrought-iron pipe* the chances appear to be nine out of ten that he will get *steel pipe* and that it will be somewhere from 5% to 15% under-weight. If the purchaser insists on full-weight pipe he will commonly be told that it will have to be made up special and that it cannot be delivered for three months, and if he insists on *wrought iron* perhaps he will get it and perhaps he won't, even if it is so named in the bill.

There is at least a well-grounded suspicion that steel

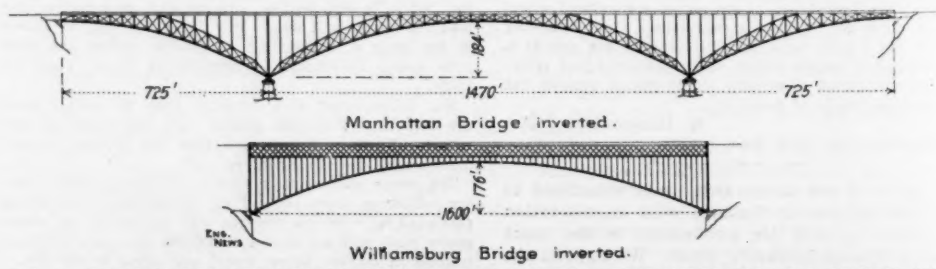



FIG. 3. DESIGNS FOR ARCH SPANS FORMED BY INVERTING THE WILLIAMSBURG BRIDGE AND THE MANHATTAN EYE-BAR BRIDGE.

pipe will rust much quicker, and a widespread opinion among workmen that it has less strength than wrought iron to withstand rough use.

I was at first at a loss to understand how steel pipe could be so much inferior to wrought-iron pipe, but I am now credibly informed that it is because the steel skelp used in pipe making is made from steel-works scrap; indeed, that it furnishes the principal outlet for getting rid of scrap or rejected metal, and that therefore little or none of this material possesses the ordinary excellent qualities of mild steel as found in boiler plates and structural shapes.

In connection with the recent work of driving test wells and borings for the Commission on Additional Water Supply, we have also run against this question of steel pipe versus wrought-iron pipe, in the tenders for material, and the skillful and experienced well-driver who is superintending this work states that if the wells are deep or the ground difficult he could hardly afford to accept ordinary "merchant wrought-iron pipe" as a gift, because of the greater chance of breakage under the severe strain of driving and pulling.

Notwithstanding the reports of so eminent an authority as Prof. Henry M. Howe, that well-made steel will probably rust out no faster than wrought iron, evidence appears to be accumulating to the effect that with such material as we get in pipe, nails and bolts, corrosion or oxidation does occur more rapidly. This makes the question a burning one among engineers and one on which we hope you can bring out some new light.

A lot of samples of defective steel pipe were recently sent me that were taken from a job of automatic sprinkler work put into a large factory at Troy, N. Y., by one of the leading contractors in the United States for water, steam and gas-piping, and on calling the superintendent to account for the fact that some of the pipe was poor and had split at the welds and had burrs inside and that the whole lot appeared to be much thinner than the standard dimensions as given in the engineering books and trade lists, we were informed that it is useless to-day to expect that ordinary pipe will be thicker than 90% of the standard dimensions and he also stated that they had practically been driven into the use of steel pipe and of accepting a moderate amount of under-weight because they could not otherwise obtain reasonably prompt deliveries; that they had to rely on their own inspection while cutting, threading and erecting to guard against imperfections, and that as to steel versus wrought iron they found about as much favorable opinion in favor of one as of the other.

I thereupon asked the engineers of our Insurance Inspection Department to look into this question, with a view to maintaining a proper standard of work under contracts for automatic sprinklers and have myself been making a few inquiries.

Some diversity of opinion is found, but there is a general concurrence among the replies, some of which I quote below, and it is plain that engineers should be on their guard and that further investigation will be very useful.

The factory manager of one of the largest contractors for pipe work in the United States answered that he believed 85% of the total amount of wrought pipe now being used in this country came from the United States Steel Corporation; that most of the pipe now being used is made of steel; that it is rather difficult to get wrought iron because the mills are slow in delivering and apparently try to make it as inconvenient as they can for the purchaser who insists on obtaining wrought-iron pipe; that the cost of wrought-iron pipe is about 2% greater than for steel pipe. He states further that a few years ago when pipe was selling at very low prices by the foot, he had much difficulty in obtaining satisfactory weights, and about two years ago he began a systematic weighing of the pipe received, and that since that time by means of judicious grumbling he has been able to obtain pipe that seldom runs more than 5% under-weight, 3% underweight being the common figure. That steel pipe appeared conspicuously as an article of commerce hardly more than ten years ago, and at first there was some difficulty and complaint about threading it; that the dies for doing a neat and rapid job on steel pipe have to be different from those best adapted for wrought-iron pipe, requiring a sharper angle on the cutting edge. He feels very confident that the mills endeavor to sell steel pipe whenever possible, and he believes that the steel pipe is more easily rolled and gives the manufacturer a greater profit.

The statements from two large users of pipe in steam and water fitting, one of whom has hundreds of workmen on pipe contracts scattered all the way from Boston to beyond the Mississippi, may be summarized as follows:

Probably somewhere in the neighborhood of 70% of the wrought pipe is manufactured by the United States Steel Corporation. Of all the steel pipe made it is probable that the United States Steel Corporation makes nearly 90%, and it is reported to be the expectation of the representatives of that corporation that within a few years there will be nothing but steel pipe made by any of their mills. Most of the independent concerns outside of the United States Steel Corporation make only pipe of small size, up to say 2½ ins. diameter.

The United States Steel Corporation is understood to make nothing but steel pipe, but would quote on iron pipe at probably 15% increased price, although their competitors who make iron pipe are obliged to sell it ordinarily at the same price at which steel pipe is sold by the United States Steel Corporation, even though the iron skelp costs them more than the steel. Usually merchant pipe runs not more than 10% under-weight, and those in the trade do not expect to-day that any pipe will be full-weight, and understand that full-weight pipe would probably have to be made up on a special order.

Pipe of a given nominal size which shows nearly the standard weight, is sometimes found to be of excessive diameter and thin, and when threaded shows the marks of the cutting tool on the inside of the pipe, which being noticed leads to its rejection.

It is stated that notwithstanding every piece of pipe shipped from the tube works is supposed to be tested to 300 lbs. pressure before it leaves the works, they often find pipes cracked along the weld. Possibly this cracking occurs during handling and its cause may be traced to the skelp having been cut a little narrow so that the edges were not forced together in the welding process sufficiently to become thoroughly cemented.

We are also told that butt welded pipe is probably less strong than lap-welded, and that although formerly the dividing line between butt-welded and lap-welded pipe was at 1¼ ins. and 1½ ins., respectively, the 1½-in. being lap-welded, to-day it is probably rare that one would obtain a lap-welded pipe of so small size as 1½ ins. unless it had been drawn down from a larger size, and that pipe as large as 3½ ins. is understood to be made often now by the butt-welding process; hence the sizes of from 1½-in. to 3-in. as made to-day, may be less strong than as formerly made.

One of the engineers of the Factory Mutual Inspection Department has a friend in the pipe business from whom he has obtained the following information:

In our business pipe is pipe as long as it is the right size and can be cut and threaded.

Either "merchant pipe" or "full-weight pipe," of either iron or steel can be had in the market. In fact, pipe sold as "merchant pipe" by several of the manufacturers comes within 5% to 6% of the published weights and is accepted at the United States navy yards for full-weight pipe.

Manufacturers when making prices commonly take the option of filling the order with either the steel or the wrought iron, as is convenient to them. Since steel pipe is cheaper to make, it is an object to them to sell steel pipe in place of iron, especially when the price of iron skelp is high, and it is hard to tell what you do really get.

In the effort to find out if one could really obtain wrought iron pipe when he wanted it, our engineer-inspector has compiled the following list of manufacturers from whom "real wrought-iron pipe" can be obtained, but it is to be noted that nearly all of these make both wrought-iron and steel pipe, and that most of the concerns named in this list make only the small sizes. We have been told that A. M. Byers & Co. and the Cohoes Tube Co. make only the wrought-iron pipe. Although the Reading Iron Works is given in the following list as among the independent concerns, it is supposed that it has some community of interest with the United States Steel Corporation. This list is arranged alphabetically and is presented provisionally, and can doubtless be extended:

A. M. Byers & Co., Pittsburg.
Cohoes Tube Co. (small pipe only), Cohoes, N. Y.
Conshohocken Tube Co., Conshohocken, Pa.
Harrisburg Pipe Co. (small pipe), Harrisburg, Pa.
La Belle Iron Works, Steubenville, O.
Reading Iron Works, Reading, Pa.
South Chester Tube Co. (4-in. and larger) South Chester, Pa.
Spang, Chalfant & Co., Pittsburg, Pa.
United States Tube & Iron Co., Pittsburg, Pa.
Weston Tube Co. (small pipe), Kewanee, Ill.
Wheeling Steel & Iron Co., Wheeling, W. Va.

I hope that by encouraging discussion of these matters in your columns, you can bring out some up-to-date information from pipe users that will be instructive on:

(1) The relative merits of wrought-iron pipe and steel pipe.

(2) The physical properties of the material used in the manufacture of pipe as affecting the strength of the pipe against bursting and for columns and as girders.

(3) The chemical composition of the metal in comparison with that used for structural steel and boiler plate.

(4) Observations on the relative life of wrought-iron and steel pipe under corrosive influences.

(5) The safe working strength of commercial pipe and fittings.

Some of these questions would form interesting thesis work for the bright young men of our technological schools, and others can best be answered out of the experience of your readers.

John R. Freeman, M. Am. Soc. C. E.

Banigan Building, Providence, R. I., Sept. 23, 1903.

**(We welcome this suggestive and instructive letter and invite others to follow Mr. Freeman's example and contribute information at their command. It so happens that certain phases of the subject are discussed in another paper, printed elsewhere in this issue, and commented on in our editorial columns.—Ed.)**

### Notes and Queries.

We have received a communication from the Abner Doble Co., of San Francisco, Cal., in reference to the letter from Mr. Geo. J. Henry, Jr., which appeared in our issue of Sept. 17. Inclosed with this communication is a copy of a letter from Prof. Samuel Storrow to the Abner Doble Co., which says in effect that no official competitive tests were made by him on the high-head impulse wheels at the Mill Creek station of the Edison Electric Co., of Los Angeles, Cal. The letter of the Abner Doble Co. also states that no authoritative competitive tests were made by Professors Storrow and Hoskins. Certain private (not authoritative) tests made by them on the two wheels are, according to Prof. Storrow's letter, not fairly comparable, as they were made under conditions which were not identical. The value of 86.2% for the efficiency of the Pelton wheel was obtained by Prof. J. N. Le Conte and Mr. G. J. Henry, Jr., in a private test conducted in the interests of the Pelton Water-Wheel Co. This test is fully reported in a paper read by Messrs. Henry and Le Conte before the American Institute of Electrical Engineers on Sept. 25. We expect to give an abstract of this paper in a future issue.

## PROGRESS REPORT OF COMMISSION ON ADDITIONAL WATER SUPPLY FOR NEW YORK CITY.*

We present herewith in accordance with your request an outline of the work undertaken and the progress made to date pursuant to our appointment of Dec. 16, 1902, and instructions received since. While we have already reached certain general conclusions, our final conclusions as to many details must depend upon surveys, designs and estimates not yet completed; for our work has been and still is largely that of balancing one project against another in the effort to review all of the reasonable possibilities for enlarging and improving the water supply, and finally to recommend that which appears best or most expedient.

Prior to Dec. 16, the date of our formal appointment, several informal conferences had been held and matters so outlined that the work of the commission was begun promptly, and such progress has been already made that we expect to present our final report on Nov. 15, 1903, the date originally prescribed. We are now enabled to present the following conclusions:

(1) That notwithstanding the most favorable anticipations relative to the prevention of waste, the immediate beginning of the construction of a large additional supply is urgent.

(2) We were advised at the outset by the Corporation Counsel, that owing to probability of litigation and consequent delay we should not consider any streams flowing from New York into a neighboring state or waters outside this state.

We find that it is possible to obtain an adequate additional supply of excellent water from streams lying wholly within the state of New York.

(3) It is feasible to secure an additional supply, larger than the present Croton and Bronx supply plus the present Brooklyn supply, capable of gradual development up to 500,000,000 gallons per day, from the following sources:

(a) From certain of the eastern tributaries of the Hudson; namely, the upper Fishkill, the upper Wappinger and the upper Roeliff Jansen Kill.

(b) From a portion of those just named in combination with the head waters of Esopus Creek, on the easterly side of the Catskill.

(c) By pumping and filtering water taken from the Hudson River, six miles or more above Poughkeepsie, or near Hyde Park; the flow in time of dought to be reinforced from storage reservoirs to be constructed in the Adirondacks for the purpose of preventing the up-flow of salt water to the intake.

All other possible sources have been considered, but none possesses the advantages of those named.

Each of these sources involves some special difficulty, and each has some advantage in location, quality or expense. Our preliminary estimates show no great difference in total cost.

We are unanimous in our preference for the upland water to that of the Hudson, although the Hudson can be made pure and palatable by filtration and should be regarded as in reserve for the more remote future.

We recommend a gravity supply from the sparsely inhabited uplands immediately north of the Croton Watershed and in the Catskills, but pending the completion of surveys and the comparison of detailed estimates we are not ready to state which of the storage reservoirs should be first built or which of the watersheds of the Fishkill, Wappinger or Esopus creeks or the Jansen Kill should be first developed.

We have had surveys made both for a high level aqueduct delivering water at nearly 300 ft. above tide at the

*The Commission consists of Prof. Wm. H. Burr, Chairman, Rudolph Hering and John R. Freeman, Mems. Am. Soc. C. E., is addressed to Mr. Robert Grier Monroe, Commissioner of Water Supply, Gas and Electricity, and is dated Sept. 17, 1903. The map presented herewith is unofficial, and is designed to show the general location of proposed sources of supply.

northern limit of the city, and for a low level aqueduct delivering at the same elevation as the present Croton supply, in order to study their relative costs, and we are unanimously of the opinion that the first additional supply introduced should be delivered at the high service elevation, which can be accomplished without pumping.

Manhattan has now two low-level aqueducts and two large low-level distributing reservoirs. The high service demands are growing rapidly, and in some of the higher parts of the city a heavy burden of expense in the aggregate is thrown upon owners or occupants in the necessity for house pumps. Under a lax administration, the economy and repair of a pumping plant may suffer far greater impairment without attracting public notice than a gravity aqueduct system; and there are elements of safety in a high-pressure gravity supply for use in connection with special fire protection mains in some of the extra hazardous conflagration districts.

(4) It appears possible to obtain from the upper waters of the Fishkill, Wappinger, Jansen Kill and Esopus creeks by means of storage in the great reservoir sites now under survey, 500,000,000 gallons daily, or as much thereof as needed, all of which will flow by gravity at such an elevation as to be delivered at the city limits at about 300 ft. above tide water, or nearly as high as the present high tower service, without pumping.

(5) By a judicious mingling of the hard water of some of the above gravity watersheds with the soft water from others, the combined supply from these elevated sources will be fully as soft as the present Croton supply. This matter, however, requires further study of the effect of spring floods upon the softness and turbidity of the stored water before exact figures can be given.

(6) After a full consideration of economy and expediency, in view of the fact that any possible aqueduct line must be largely in tunnel and must be built of size suitable for future needs, delivering at times more rapidly than at the average rate, we recommend that the next aqueduct be built of a capacity of 500,000,000 gallons per 24 hours.

(7) We strongly recommend that the city should at once begin the construction of filters, both for the Croton water and for all other waters taken from surface streams, and we have surveys and plans for filtration in progress for each and all of the new sources recommended. Further plans and estimates must be completed before we are ready to state our conclusions as to the most favorable sites for filtration plants.

(8) We consider that the surface streams that form so large a part of Brooklyn's present supply should either be filtered, as already in progress, taking the most polluted first, or that the ground water being naturally filtered should be taken, so far as practicable, by means of carefully planned wells or intercepting galleries of a design yet to be perfected.

(9) The one great natural obstacle which limits the date of the early introduction of an additional supply from any and every adequate source within the state of New York, is found in the long deep tunnel under the mountains and hills east of Peekskill. There are also some high dams needed in some of the projects, and it now appears that under the most prompt action and with the best of management fully five years will be required after construction is begun before the water of any additional up-country supply can be delivered in New York.

(10) For nearly its entire length, the aqueduct has to be built of full size for its ultimate 500,000,000-gallon daily capacity, and although but two reservoirs be built at first, it is most expedient to construct them of full capacity and with their dams at full height. Therefore, the first instalment of the completed work will be the most expensive, and after this from time to time as the city grows and its population increases, other reservoirs can be added or extensions made to more distant points at comparatively small outlay.

The preliminary estimates indicate that the expenditure for the first instalment of the increased supply will be not far from $50,000,000. This will complete the necessary dams, reservoirs, aqueducts, tunnels and filters, and acquire the necessary lands and rights sufficient for a daily supply of upward of 200,000,000 gallons.

This is a large sum, but for comparison we may note that the Boston Metropolitan district has, within the past eight years, expended about $20,000,000 upon an additional supply of about 110,000,000 gallons daily, for a total population now of about one million.

The New Croton Aqueduct, about 30 miles long, built mainly between 1886 and 1892, cost complete about $20,000,000. The Jerome Park Reservoir will cost probably $11,000,000, and the New Croton Dam and accessory works probably $10,000,000; which, exclusive of recent reservoirs in the Croton watershed and other appurtenant works, brings the total up to more than $40,000,000 for the extension of the Croton system since 1885.

The revenue of the water department for Greater New York collected in the current year will be upward of $9,000,000, and will normally show an increase of $500,000 per year.

(11) Our Long Island investigations, so far as completed, indicate that the water resources of Long Island should be utilized to the greatest possible extent for the public supply of communities on Long Island, including Brooklyn, and that the use of this water in general will be cheaper than to bring the additional water wholly from the north; moreover, its entire independence is a safeguard against disaster, but the greatest amount of additional water that can be had here without extensions into Suffolk County, may not be more than enough to take care of the natural growth of Brooklyn and the other Long Island communities until the water from the north becomes available, or until it becomes demonstrated to the public that ground waters from Suffolk County can be taken without real injury to local interests.

(12) The need of a further water supply for Queens Borough is urgent, and we have recommended the development of the present ground water supply on lines already undertaken by the department as submitted to the Board of Estimate and Apportionment.

(13) The Borough of Richmond has no adequate supply available within its own limits and its needs are also urgent. The increased supply for this borough may in time be taken from the water supply of Brooklyn when increased by the additional supply to be secured from the north. We approve of the measures now being taken to secure in the meantime the needed water from the nearest available supply on the mainland of New Jersey.

(14) We find no possible quickly available and near source, or anything that would be worth the trouble or expense of development as an interim supply to make good the present excess of consumption in Manhattan and the Bronx over the recorded actual yield of the present watersheds in a year of extreme drought.

We therefore urge water-waste prevention in strongest terms, as the best and cheapest safeguard; and, after having seen the result of the earnest efforts at waste prevention made in certain typical districts during the past season, by means of district meters and house to house inspection, urge the continuance and extension of this kind of work. The members of this commission desire to record their unqualified recommendation of the immediate extension of metering to classes of buildings not yet included in the ordinances, as one of the best means of permanent prevention waste.

### Progress of the Work.

The instructions of this Commission in brief were, to investigate and report upon:

(1) The quickest and best method of reducing the waste of water.

(2) Recommendations of the most practicable means for improving the quality.

(3) The pressure and means of distribution for attaining the highest practicable degree of fire protection.

(4) The probable future consumption of water, having in view the natural increase of population, increase in per capita use, and the prevention of waste.

(5) The future sources of supply most available in view of cost and quality, with estimates of their cost.

(6) The feasibility of a temporary and supplementary supply.

Appreciating the large amount of important work to be done in a short time, this Commission, immediately after organization, divided the main portion of its work into six departments, and for taking charge of the detailed studies sought engineers of extended experience on similar recent large water-works constructions near New York, Boston, Philadelphia, North-Eastern New Jersey and elsewhere. These departments are: (1) Eastern Aqueduct and Reservoir Department. (2) Catskill Department. (3) Filtration Department. (4) Chemical and Biological Department. (5) Long Island Department. (6) Department of Pumping.

### WATER WASTE PREVENTION.

We were relieved from the necessity of establishing a department of studies of water waste by the fact that this work had already been begun in the Borough of Manhattan and the Bronx by Mr. Nicholas S. Hill, Jr., Chief Engineer, and that it could be best carried on in connection with the regular work of the department having charge of valves, meters and distribution pipes. We are also informed that similar work has been inaugurated in the Borough of Brooklyn by Mr. I. M. de Varona, Chief Engineer. Pursuant to your desire for co-operation we made ourselves familiar with Mr. Hill's plans and had frequent conferences regarding the results of pitometer measurements and the house inspection, sewer inspection, etc., of typical districts; approving fully of the plan of work. The results have been already set forth in reports to you by Mr. Hill from which we draw the following conclusions:

(1) The leakage from the mains is much less than hitherto supposed. The distribution system of New York needs many new gate valves and hydrants to bring it into satisfactory condition, but the deterioration of the street mains is not such as to require extensive renewals to prevent waste.

(2) The main sources of waste are leaky fixtures and defective plumbing design. The loss from leaky fixtures probably exceeds 15% of the total supply in the Manhattan and Bronx Boroughs or upward of 40,000,000 gallons per day. The reduction of waste due to leaky fixtures can be accomplished by constant inspection, and that certainly has been done during the last twelve months as demonstrated by the returns of the chief engineer, submitted to us. The permanency of this reduction can only be secured by the continuance of the system of house to house inspection which has been so effectually conducted in these investigations in typical districts. Under the defective design of plumbing, hot and cold water pipes are usually placed adjacent to each other without proper circulation, requiring the waste of large quantities of water before securing either the hot or cold water desired. We do not believe it feasible to reduce materially the extravagant use of water due largely to defective plumbing design.

(3) The reduction of all waste is effectively aided by the use of meters, which should be extended to other classes of buildings than those now metered. We recommend that all buildings over five stories in height be metered at the earliest practicable date. All meters should be owned, installed and maintained by the city and tested at regular intervals in order to secure reasonably effective service from them.

(4) One important result demonstrated by these investigations is the absolute unfairness of frontage charges, leading to marked inequality of burden on users in different portions of the city. This is most manifest in connection with the large apartment houses covered by our recommendation to extend meters to all buildings over five stories in height.

(5) These investigations have also demonstrated the necessity of considering the great transient population of the city when accounting for the per capita consumption. The per capita consumption in one residence and hotel district was found to be 175 gallons based upon the resident population and 121 gallons when based upon the combined resident and transient population. A district embracing a portion of the downtown office section disclosed a per capita consumption of 860 gallons per day when based upon resident population and 83 gallons per day when based upon combined resident and non-resident population.

(6) These investigations indicate that the possible saving by reduction in waste and extravagant use will not more than offset the natural increase in demand due to growth of the city, and hence the construction of an additional supply should be undertaken at the earliest practicable moment.

### EASTERN AQUEDUCT AND RESERVOIR DEPARTMENT.

The Aqueduct and Reservoir Department, east of the Hudson River, began operations on February 4. These operations have thus far included the survey of about 125 miles of aqueduct locations, extending from the Jerome Park Reservoir to Roeliff Jansen Kill. These locations comprise 80 miles of line for aqueduct following the contour of the ground and designed to be built of masonry in open trench and covered by earth, about 40 miles of tunnel and about five miles of large steel pipe siphons; a part of the above being alternative projects. In addition, a large amount of open channel work has been surveyed.

The results of these field operations have been mapped to a large scale in permanent form suitable for future use.

A line of bench levels was run parallel to the aqueduct line, reference points were located at frequent intervals, and side topography was taken by the location party as the work advanced, so as to permit shifting the line to one side or the other, in order to gain economy in final studies. The surveys were tied to the triangulation points of the U. S. Coast and Geodetic Survey and numerous places.

In the effort to fix upon the best possible location, the field parties are now retracing certain lines where alternative routes appear feasible and where the best location was doubtful.

HIGH LEVEL RESERVOIR STUDIES.—Near Stormville and Billings on the upper Fishkill, reservoir sites of about 4½ and 2½ square miles have been surveyed, contoured and plotted and the capacities corresponding to various elevations of water surface computed. Jet borings are now in progress at the sites of the dams to determine the character of underlying strata and bed rock.

At Hibernia, on the upper Wappinger, surveys are being made of a promising site for a reservoir much larger than the new Croton reservoir, requiring one main dam and a dike. Borings to determine the character and position of bed rock will soon be completed. Approximate estimates of capacity and cost of this reservoir have already been made.

Near Clinton Hollow, about twelve miles northeast of Poughkeepsie, on the upper Wappinger, surveys for a high-level reservoir have been begun and will require about six weeks for completion.

Near Silvernails, on the upper Roeliff Jansen Kill, about 25 miles northeast of Poughkeepsie, an excellent site for a high-level reservoir, covering about nine square miles, has been surveyed, contoured and plotted, with dam site plans in detail. These plans and estimates are well advanced toward completion.

Surveys of large terminal reservoirs have been made, and a contour plotting completed ready for studies of excavation and embankments and estimates of cost.

LOW-LEVEL RESERVOIR STUDIES.—In connection with the low-level aqueduct lines, extensive surveys cov-

ering 17 square miles have been made of a possible site for a great reservoir near Fishkill, and a large portion of the work is plotted. This survey presented unexpected difficulties, among which are the many thickly-wooded swamps. These swamps are all surveyed and the depths of muck determined.

On the Wappinger, surveys covering ten square miles have been made for a low-level reservoir, and several dam sites have been studied. Contour plans of this reservoir site have been drawn.

Other surveys have been made in an extended study of other possibilities for large reservoir sites at suitable elevations east of the Hudson River, by means of the maps of the U. S. Geological Survey and reconnoisance in the field.

Comparative designs and estimates have been made to determine the best and most economical sections for aqueducts and tunnels, the best methods of tunnel lining and other structural details from which close estimates of cost will be soon made or from which contract drawings may be prepared.

DIVERSION DAMAGES.—The data for estimates of damage to water powers and factory sites on each of the streams where a diversion of water is proposed have been collected. Nearly every factory has been visited and notes made of its water fall, power and use of water. The estimates of money value of damage sustained by the diversion have as yet been only approximated, but at what is thought to be an outside amount.

### CATSKILL DEPARTMENT.

The Catskill Department was organized early in March. The principal work of this department has been done in the Esopus Valley, but it extended into the upper portions of the Schoharie and Catskill Valleys. The surveys made two years ago under the former Department of Water Supply were availed of as far as possible.

Surveys in much detail have shown that near Bishop's Falls on the Esopus, a dam can be built higher than heretofore supposed practicable, which will flood an area of more than 14 square miles and form a storage reservoir of more than double the available capacity of the new Croton reservoir when completed, and of admirable form for holding the water several months in storage in transit from the river to the aqueduct, thus favoring purification, sedimentation, bleaching and the destruction of pathogenic germs.

In this work a stadia survey was made of nearly ten square miles of more or less broken and wooded land, together with a less complete survey of six square miles more. Contour and other maps of this site have been completed.

The Ulster & Delaware Railroad runs through this reservoir site and two alternative surveys for its re-location, comprising 28 miles of location, have been run with such full detail that quantities and costs can be closely estimated. Nearly 1,000 test pits have been dug on the reservoir site in order to determine the character of the ground and the requirements for stripping or covering areas of peat or muck.

Other reservoir sites in the Esopus Valley have been located at Cold Brook, Lake Hill, Wittenburg, Shandaken and Big Indian. Careful stadia surveys have been made of each, covering more than five square miles within projected flow lines; five dam sites have been studied, all property lines have been located, and the necessary plans made for estimating quantities and costs of each.

A tunnel line 13 miles long crossing under the Catskill Range, from the Schoharie watershed into the Esopus Valley, has been reconnoitered and surveyed, and stadia surveys have been made for four reservoir and dam sites in the upper Schoharie watershed on the northwestern slopes of the Catskill Mountains, available for supplementing the Esopus in the more distant future. The computation of contents and the maps of these Schoharie reservoir sites are not yet completed, but they are well advanced.

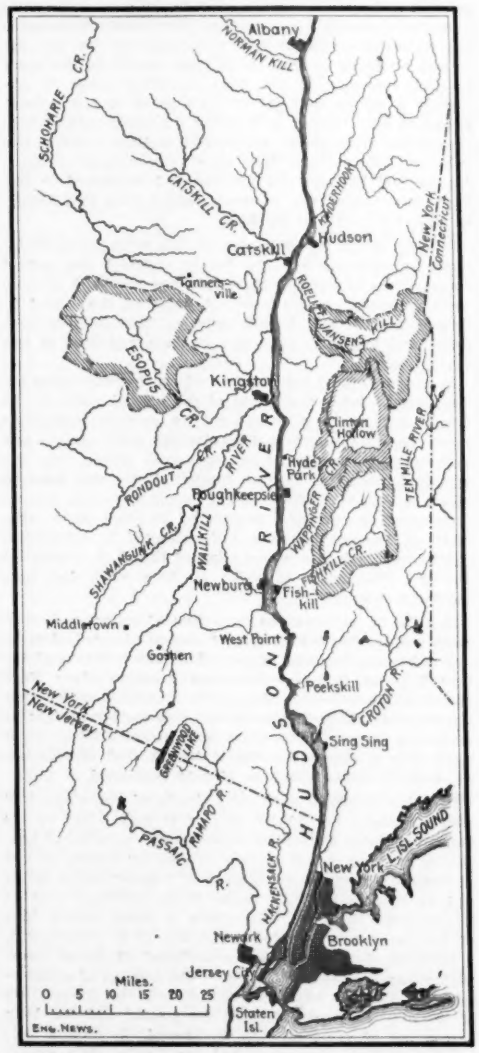
For the sewage disposal of the villages of the Esopus region, mainly summer colonies, careful studies have been made, together with a detailed sanitary inspection of the whole region. Detailed plans for sewage disposal works have been made for the villages of Pine Hill, Chichester and Phoenicia, with estimates of cost. On the Schoharie watershed, surveys have been made for a trunk sewer for the removal of the sewage of the entire valley to a point below the lowest reservoir. On Catskill Creek, considerable work of a similar kind has been done.

RAINFALL STUDIES.—Much study has been given to the rainfall of the Catskill Mt. region to determine its volume in comparison with that of the Croton watershed. This has involved a painstaking scrutiny of all available records, correction of errors and the establishment of ten new stations, together with the observations of the latter. A river gaging station was established on the Esopus near the principal dam site, in co-operation with the U. S. Geological Survey, and daily observations were maintained. Two rain gages have been established high up on the mountain side in representative locations on opposite sides of the valley for comparing the rainfall on the heights with that in the valley. The indications are that it is materially larger, but the observations and studies so far made are not extended enough to warrant conclusions and, like many other series of meteorological and hydrographic observations begun by this Commission, should be taken in charge by the regular engineering staff of the Department of Water Supply, Gas and Electricity, and continued for a series of years.

### FILTRATION DEPARTMENT.

Surveys were begun on Feb. 19, 1903. At first the work of the entire force was directed toward the discovery, surveying and mapping of all areas along the aqueduct routes lying at suitable elevation and of sufficient size for filter sites.

Following the map studies and a reconnoisance, lines of bench levels starting from the precise level benches of the U. S. Government, near Tarrytown, Fishkill Landing and Poughkeepsie were run for more than 50 miles, on circuits through the territory in which filter sites were sought. Based on these levels, stadia surveys were made of the available areas near Brinkerhoff, Fishkill, Stormville, Freedom Plains, La Grangeville and Pleasant Valley in Dutchess County, Elmsford, Scarsdale, Greenville and other points in Westchester County, in order that comparative estimates of cost and construction could be made and the most economical sites selected. This work involved the survey of 42 square miles of territory, the




Unofficial Sketch Map Showing the General Location of the Proposed Additional Sources of Water Supply for New York City.

running of 290 miles of traverse lines, and about 40,000 locations with elevation and position, sufficient points being taken to admit of locating contours at 5-ft. intervals. Eighty-eight survey sheets have so far been prepared and are now in use in making the final studies.

Filtration projects suitable for each of four independent schemes have been worked out covering all the projects for upland and Hudson River waters and for the filtration of the Croton supply.

Studies for improved methods of handling the sand and otherwise lessening the cost are now well advanced, designs of standard details such that full working drawings can be promptly made for filter construction are now in preparation and a close estimate of cost will follow.

We hope to be able on Nov. 15 to present complete plans for a first instalment of filters, of 50,000,000 gallons per day capacity.

### CHEMICAL AND BIOLOGICAL DEPARTMENT.

This department was organized in the later part of December, 1902, so as to be in readiness to collect samples of the winter or spring flood waters in all streams available for water supply.

The sanitary investigations extended from the Narrows to the headwaters of the Hudson along its principal affluents. Samples were taken from the principal eastern tributaries all the way from the Croton watershed north to Schroon Lake in the Adirondacks. On the western bank, the Esopus, Roundout and Catskill creeks were investigated. The streams on Long Island east of the present water shed of the Brooklyn supply were also frequently sampled.

The entire territory examined covered over 15,000 square miles. The magnitude of the work may be appreciated by the statement that between Jan. 1 and Sept. 1, over 4,000 samples of water had been collected and examined.

In this department we availed ourselves of the excellent facilities of the Mount Prospect Laboratory of the Department of Water Supply, Gas and Electricity and all the sanitary studies of the Commission were made with the cordial and efficient co-operation of its staff.

A field laboratory was established at Poughkeepsie and equipped for the complete physical and bacteriological examination of water, but all chemical and microscopical analyses were concentrated at the Mt. Prospect Laboratory.

The instructions of the Commission to this department covered the making of such investigations and analyses, chemical and biological, as were necessary to determine the sanitary character of the various waters available for the supply of New York. As the work progressed its scope was extended to cover a study of the physical characteristics of the soil within the Long Island watershed from which Brooklyn is supplied, including soil moisture and its relation to rainfall and depth of ground water. So far as possible existing facilities and employees already in the service of the city were utilized.

INVESTIGATION OF THE PRESENT WATER SUPPLY.—The investigations of the present supply have been carried on with respect to the immediate needs of the department, but various compilations of analyses have been made, to serve as a basis of comparison with the additional sources considered.

Among these compilations may be also mentioned a study of the typhoid fever statistics from 1868 to 1903, and a comparison with the rainfall and the past extensions of the present water supply. It is a matter of congratulation that the typhoid fever death rate in New York has been found much lower than in most large American cities.

Croton water has often been turbid during the past few years and occasionally malodorous, but nevertheless appears to be generally satisfactory from a sanitary standpoint. The odors are due not to pollution, but to the effect of microscopic organisms which grow at certain seasons in the reservoirs and especially in the distributing reservoirs in Central Park. The unsatisfactory physical condition of the Croton water has led to the expenditure by citizens of hundreds of thousands of dollars in household filters, stills, etc., and to large annual expenditures for spring water, all of which would be rendered unnecessary by public filtration.

The Brooklyn water has also been carefully studied. It is generally clear, but occasionally turbid after heavy rains.

The hardness of both the Croton and the Lond Island water average about 40 by the ordinary scale, but the Brooklyn water is somewhat less desirable than the Croton for steam boilers, because of the infiltration of sea water into some of the driven wells near the shore. The population in some portions of the watershed now drawn on is rapidly increasing and the surface supplies, from brooks and streams, are being exposed to increasing dangers from pollution.

SANITARY STUDIES FOR THE ADDITIONAL SUPPLY.—There is a growing demand for improvement in the quality of public water supplies and our studies were entered upon in the conviction that the new supply must be better than present supplies. The following qualifications were kept in view as necessary throughout the work:

(1) Absolute freedom from pollution.
(2) Freedom from odor, turbidity and noticeable coloring matter.
(3) Softness.
(4) In the case of ground water, freedom from large amounts of iron.

Four distinct lines of investigation were carried on in the field:

(1) A study of the watersheds and the water samples from the various streams considered as probable sources of supply.
(2) The study of the water of the Hudson with reference to its filtration.
(3) Study of the ground waters on Long Island.
(4) Investigations in soil physics.

STREAM INVESTIGATIONS.—Stations were established on all of the important streams and daily samples were collected and sent in at the end of every week to the Laboratory at Poughkeepsie, where they were examined for turbidity, color, alkalinity and hardness. At the time of collecting each sample, the stage of the river was observed and the meteorological conditions were noted. Rain gages were placed at ten new points scattered over the various watersheds. From time to time samples for chemical analyses were collected at the most important points, comprising 35 stations in all, located at 18 different streams.

In addition to the analyses of water samples from these regular daily stations, investigations were made of the watershed lying up-stream from points where samples were collected. These investigations differed much in their completeness, according to the probability of the stream being selected as a source of supply. As the work progressed and the exact field for selection of sources of supply was narrowed, more attention was given to the Fishkill Creek, Wappinger Creek, Roeliff Jansen Kill and Esopus Creek.

The waters of the Roundout and Esopus are of excellent quality, being light colored and very soft. After rains they become slightly turbid from the effect of a few local clay deposits which may be readily controlled at comparatively small expense. The Esopus water is better in quality than that of the Roundout. In fact, the investigations thus far made indicate the Esopus to be superior in quality and better adapted for general public and domestic use than any other untreated natural water examined. It has less than half the hardness of the present Croton supply, is tasteless, odorless and practically colorless and is clear except for the cloudiness observed after rains which would disappear after storage. Filtration would render this water well nigh perfect for purposes of public supply. The water of the Catskill Creek has been found clearer than that of the Esopus, but it contains more calcareous matter and has a hardness which exceeds that of the Croton. The water of the Schoharie Creek is soft and has little color.

The Roeliff Jansen Kill, Wappinger Creek and Fishkill Creek are practically unpolluted, clear, tasteless, odorless and with low color, but rather hard at times. This region is overlaid in places with limestone, and much variation in hardness is found in streams near together, obviously the hardness of the water flowing in summer, which is mainly ground water, does not represent the hardness that would be found in the same water collected during the spring freshets in storage reservoirs, which has not percolated through the ground. The studies to determine the probable hardness of reservoir waters are not complete, but we are justified in stating that by a mixture of these waters from the Esopus the hardness of the whole would not exceed that of the present Croton supply. If the entire additional supply were taken from the eastern streams alone the hardness would be somewhat greater.

HUDSON RIVER WATER.—The average of many analyses shows that the hardness of the water at the proposed future intake between Greer Point and Hyde Park would be substantially the same as that taken from this river at Albany and that this water could be made at least equally satisfactory by filtration. The additional pollution received below Albany is more than off-set by dilution from the volume of water received from tributaries entering below.

The pollution of the Hudson was carefully studied statistically by making estimates of the city and country population per square mile at various selected points between Glens Falls and New York Harbor, and by ascertaining the typhoid statistics of all the cities along its course, the amount of sewage and factory waste. Samples for examination have been taken daily since May 1, at Mechanicsville, Cohoes, Catskill, Saugerties, Kingston, Hyde Park, Poughkeepsie and Garrison. Daily bacteriological examinations have been made at Poughkeepsie and weekly chemical analyses have been made upon the samples collected between Newburgh and Catskill. Samples have also been taken to ascertain the efficiency of filters receiving Hudson River water at Poughkeepsie.

By adopting proper precautions the water between Poughkeepsie and Hyde Park would be rendered perfectly safe for domestic use. It would have an average hardness probably a little greater than that of the present Croton supply.

SALT WATER IN THE HUDSON.—Much time was devoted to finding the limit of up-flow of salt water in the Hudson. The present season has been less favorable for this purpose than if there had been less rain, but many series of analyses have been made and the tidal phenomena have been observed and studied. Automatic tide gages were located at Yonkers, Oscawanna, West Point, Poughkeepsie and Rhinecliff.

The preliminary review of these and other data gives reason for belief that the chlorine never extends up the river beyond Greer Point even in the driest year.

GROUND WATER STUDIES.—While the many analyses of the ground water supplies of Brooklyn and Queens furnished the basis of these studies, various samples from other portions of Long Island were analyzed and comparison made with the density of population in the regions surrounding the wells from which the samples were taken. Samples from the thickly populated portions of Brooklyn were also analyzed for comparison, and the results show that many ground waters collected by means of driven wells from regions which have a large population and which might be considered unsatisfactory on chemical analyses are satisfactory bacteriologically and may be safely used for drinking purposes. This is due largely to the sandy character of Long Island, which renders it an excellent filtering medium.

Investigations were made to determine the limit at which pollution will extend from large amounts of faecal matter deposited at a depth of 6 or 8 ft. below the ground, as in privy vaults. Other experiments have been made to determine the decrease of bacteria from the surface of the ground downwards in the soil.

The studies of the water in Suffolk county show it to be very similar in character to that of the present Brooklyn supply.

In the study of soil physics over a thousand samples have been collected and dried and their moisture determined by differential weighings. These were collected at a number of representative places with the water table at different distances below the surface and the results, although not yet completely worked out, are exceedingly instructive.

## LONG ISLAND DEPARTMENT.

This department was organized in March. The chief object sought was definite information about the underground sources of water available on Long Island, through driven wells or wells of other types, intercepting galleries or other means. A brief review of the situation suffices to show that the surface streams from which the larger portion of the supply is now taken will become more and more polluted as the population increases and spreads into the suburbs, and must be filtered to be fit for use, whereas the ground water is filtered naturally. Moreover, it is desirable to learn what is the greatest possible quantity of ground water that can be taken from the present watersheds without injury to property, for this excellent water so near at hand, and stored in the great natural reservoir of the deep saturated sands is obviously a source that should be utilized to the utmost. Attention was directed to learning the topographical features of the subterranean watersheds, and the direction and velocity of flow of the deep, slow subterranean currents; for the real limits of the underground watershed in this kind of formation may be very different from that bounded by the summits of the hills.

Diligent search was made for all dug wells favorably located for observation of the height at which the ground water naturally stands. More than one thousand of these wells, scattered nearly over all parts of the island in Queens, Nassau and Suffolk counties, have been investigated, and the height of water in them measured at frequent intervals.

In sparsely populated districts where the wells were too far apart to define the height of the ground water in the intervening territory or where the surface indications gave reasons to suppose irregularities might exist, test wells of 2-in. wrought-iron pipe were driven into the saturated ground to varying depths, all the way from 20 to 100 ft. Upward of 250 of such wells have been driven, samples of the material penetrated by these wells were taken at frequent intervals and subjected to mechanical analysis through the co-operation of the U. S. Geological Survey. The height of water in these wells has been regularly observed.

In order to compare the elevations of water surface in these wells, extensive circuits of accurate levels had to be run and connected with the standard bench marks of the United States Government at the Brooklyn Navy Yard and the tide gages at Governor's Island, in order that the relations of the heights of water to sea level might be accurately known. This work will require lines of levels more than a thousand miles in length, and of this upward of 650 miles have been already completed.

In order to obtain a closer estimate of the amount of water available in addition to that already taken by the Brooklyn works, a series of observations was planned consisting of observations on the rainfall by means of two self-recording rain gages located at representative points and by three other rain gages of the ordinary type at intermediate locations. To obtain a more precise idea of the percentage of rainfall passing off in evaporation, a meteorological station was established at Floral Park, with apparatus designed to record the amount of evaporation, the direction and force of the wind, the temperature of the air, the temperature of the ground and the daily percentage of sunshine. At Brentwood, in Suffolk Co., another station was established for similar data except that the apparatus for measuring the evaporation was not duplicated. These observations were carried on in connection with the frequent observations on the soil moisture at different depths referred to under the work of the chemical and biological department. Weirs for precise water measurements were set on the principal streams and equipped with autographic registers.

MEASUREMENTS OF VELOCITY OF UNDERFLOW.—The U. S. Geological Survey co-operated generously in this work and detailed Prof. C. S. Slichter to make measurements of the velocity of the underflow. The field work has been nearly completed, but the results are not fully computed.

The results of all this work will be of great value and several of the lines of observation should be assumed by the regular forces of the department and carried on for a series of years.

The results already obtained show that the real limits of the underground watersheds are materially different from the boundaries indicated by the surface topography.

## THE DEPARTMENT OF PUMPING.

This department was organized with a view to planning the large pumping stations necessary in connection with the taking of water from the Hudson River, had that source been selected for the first development, and for studies of other pumping plants for pumping from certain promising reservoir sites that lay at an elevation lower than the aqueduct. There have been great advances in this branch of steam engineering in recent years and the comparative economy of a system involving pumping in comparison with one delivering by gravity depends largely on the design and arrangement of the pumps and the steam power equipment. As the pumping station for a supply taken wholly from the Hudson River would have necessarily contained pumping machinery of much greater capacity than any plant now in existence, it is obvious that most careful studies of this design were paramount.

As the surveys and estimates progressed and it was found practicable to obtain the additional supply now required from sources lying at an elevation such that the water can be delivered by gravity, the work of this department was largely directed to co-operation with the chief engineers in the Department of Water Supply, Gas and Electricty in improving the efficiency of the present pumping stations forming a part of the Manhattan and Brooklyn systems. The principal work thus far accomplished is the following:

(1) In connection with the building of new pumping stations, as a part of the system of additional supply, tables and graphical charts have been prepared showing the estimated cost of building and maintaining stations having a daily capacity up to 500,000,000 gallons.

(2) In co-operation with Mr. Nicholas S. Hill, Jr., Chief Engineer, a careful examination was made of the mechanical efficiency of the three pumping stations in Manhattan, at 179th St., High Bridge, and 98th St. About May 1, prior to the first of these studies, five engines, including the new Worthington engine, were kept in operation at 179th St., and one engine at the High Bridge Station in order to supply the consumption. Two of these engines having a daily capacity of 10,000,000 gallons each were found by these tests to have a slippage or loss of action in the pumps of 65% and 60% respectively, and two engines having a daily capacity of 4,000,000 gallons each were tested and found also to have some slippage. On account of the necessity for continuous running it was decided not to take the time then to test the steam engine efficiency in detail, but to proceed immediately with the repair of the pumps. Each of the 10,000,000-gallon engines was dismantled, and after the repairs were finished on all four pumps it was found that by the prevention of slippage one engine at 179th St. could be shut down all the time and that at High Bridge the greater part of the time, while the other four engines, including the new Worthington engine at 179th St., performed all the work previously performed by the six engines. This immediately resulted in a saving of about $37 per day in fuel and oil.

There will be a further saving as soon as the repair work now in progress on the steam cylinders and valve gear is completed. At the 98th St. Station, the engines had been repaired but a short time previous and were found to be in fairly good condition.

Further studies were made of possible improvements in pumping station economies by means of improved methods of firing and other changes of detail. When the improvements already begun have been fully carried out it is estimated that there will result an annual saving of not less than $20,000 in value of fuel alone and 40% in value of oil and small supplies at the three Manhattan stations.

(3) In connection with the Brooklyn pumping stations, the work of this department has so far consisted chiefly in co-operating with the chief engineer, Mr. I. M. de Varona, in the study of methods for securing greater economy in the operation of the present numerous driven-well pumping stations by operating them by electric power generated at a central power plant that may form a part of the Millburn pumping station, in assistance upon the preparation of specifications for new high-service pumps at Ridgewood, in preparing plans and specifications for the new Gravesend station and in perfecting the design of the Wantagh station. The average cost of pumping 1,000,000 gallons of water one foot high at the driven-well stations is 28 cts., and the estimated cost of pumping the same quantity with improved centrifugal pumps operated by electrical motors is 8.5 cts.

The tests of the Ridgewood station are not yet finished, but the preliminary inspections indicate that by improved methods the present cost per 1,000,000 gallons at the station probably can be decreased 30%, representing an annual saving in total cost of maintenance of $75,000.

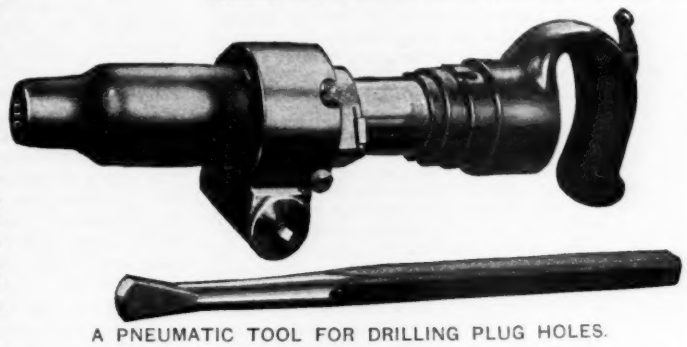
## FORCE EMPLOYED BY THE COMMISSION.

The foregoing work has required the services of 6 department engineers, 5 principal assistant engineers, 12 assistant engineers, 24 draftsmen and computers, 9 transitmen, 10 levellers, 13 rodmen, 12 chainmen, 12 axemen, 8 test-well foremen, 23 laborers, 6 stenographers and typewriters, 4 chemists, 6 biologists, 5 inspectors, 39 sample collectors and gage readers, making a total of 195.

The above represents the average force during the summer. The field force has been largely reduced since Sept. 1, by reason of partial completion of the surveys. The remaining work is largely that of the reduction of field observations, the preparations of plans and estimates of costs.

## A PNEUMATIC TOOL FOR DRILLING PLUG HOLES IN SPLITTING STONE.

In splitting quarry stones plugs and feathers are so extensively used that any machine for drilling the plug holes by power instead of by hand will find a wide field of application. A new plug drill that in appearance much resembles the common pneumatic riveter is shown in Fig. 1. It differs from the riveter in being provided with a rotating device which automatically turns the drill while the hammer is in action. The drill weighs but 18 lbs. complete, consumes 20 cu. ft. of air per minute at 80 lbs. pressure, and will drill a ⅝-in. hole 3 ins. deep in any kind of stone. The manufacturers, at our request, have had a record kept of the number of holes drilled in granite in an hour, and report as follows:



A PNEUMATIC TOOL FOR DRILLING PLUG HOLES.

One test developed 60 plug holes, ⅝ × 3 ins., drilled in 60 mins.; each drill used sinking 20 holes without resharpening; this included the lining, changing of drills, etc. On another occasion 131 holes of the same description were drilled in 140 mins. On the test of drilling a single hole we have done same in as short a time as 20 secs., but the average time for a plug hole varies from 30 to 40 secs. All of the above figures are based on the drill working in granite, in all soft stone the cutting is much more rapid.

When it is remembered that it takes a skilled man 4 to 5 mins. to drill a similar hole in granite by hand, the saving in labor effected by one of these pneumatic plug drills is apparent. The manufacturers of this plug drill are the Chicago Pneumatic Tool Co., of Chicago, Ill.

## THE CORRECT DESIGN AND STABILITY OF HIGH MASONRY DAMS.*

By Geo. Y. Wisner, M. Am. Soc. C. E.†

The developments of projects for the storage of flood waters for the purpose of irrigating arid lands in the Western states will require the designing of dams of unprecedented heights, and in view of the failure of similar structures which were considered safe by the engineers who designed them, it is imperative that the plans be based upon accurate data and upon a correct determination of the nature of the stresses to which they will be subjected. If such information does not exist, there should be no excuse for its not being obtained before final plans are adopted for all works where failure will result in the loss of either life or property.

With a correct knowledge of the weight, strength, modulus of elasticity and coefficient of expansion of the materials of construction, the relation of the stresses in the horizontal arch of a dam to those in the vertical sections, under varying water pressures and temperature strains, can be determined within reasonable limits, and the parts of the structure so designed that its safety will be beyond question.

With the facilities that are at the disposal of the engineer for determining the strength, elasticity and expansion of the materials of construction for large structures, it would be criminal negligence on the part of the designer not to make such determinations.

In the development of the theory of the laws which govern the action of forces in large dams, most of the older investigators, as well as many engineers of the present time, have assumed that a dam is a rigid homogenous monolith, and that the foundations (while more rigid and non-elastic than the structure) are perfectly elastic. In the design of some recent structures, the reverse of these assumptions has been adopted, viz: That the materials of which the structure is formed are homogenous and perfectly elastic. Both of these assumptions certainly cannot be correct, and, as similar conclusions are derived in each case, it hardly seems good policy to base the design of large structures on such precedents.

In regard to the elasticity of a completed dam, it may be stated that the materials of which it is constructed are far from being perfectly elastic and homogenous, and even if they were, the method of construction makes the dam practically a rigid structure. The faces of the dam are usually of cut stone masonry with thin joints, having a modulus of elasticity of from two to five times that of the concrete or rubble filling of which the interior of the dam is composed, thus practically forming a triangular truss, having a height of about 1⅛ its span, which for stresses up to point of rupture through the toe of the dam, may be regarded as rigid. The natural rock on which the structure should be founded is under heavy compression, and held firmly in place by the compressive forces in the earth's surface, so that elastic movement under load will be so small that the structure may be regarded as resting on a rigid base. Hence, conclusions based upon assumptions that the dam is a perfectly elastic structure, or a rigid structure resting upon an elastic foundation, will not hold true.

Under either of these assumptions, the conclusion reached, which forms the basis of analysis of stresses, is that if the resultant due to the water pressure and the weight of the dam falls outside of the middle third of the base, tensile strains will be developed in the upper face of the dam, a condition which, as dams are generally constructed, is not likely to occur before the toe ruptures and the dam slides on its base.

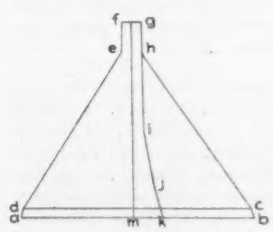
The conclusion that tensile strains will develop in the face of a dam when the resultant passes beyond the middle third, requires that the dam act as a flexible beam bending about its central line as a neutral axis, and that with the reservoir empty, there will be no pressure on the foundation at the toe, and have a maximum pressure at the heel, and with the reservoir full, there will be no pressure at the heel, and be a maximum at the toe. Under the usual form of construction of dams, the tendency to movement under change of water pressure is about the toe, instead of center of the base, and as a result the greatest stress in the structure from horizontal water pressures on the face, will be parallel to and near the back face of the dam, and the pressure on the heel of the dam will not become negative when the resultant passes the middle third, and probably will not until the toe ruptures, which result materially modifies the actual distribution of pressure on the base, from what would be the case under the assumption of perfect elasticity of structure.

The fact is that there is far greater danger of failure from shearing, or sliding on the base, than from tensile strains in the upper face, or from overturning of the structure. The force tending to produce shearing or sliding increases with the square of the depth of the water, and in straight dams, where the tensile stresses from changes of temperature may exceed the strength of the masonry, this source of danger is a serious one, and practically limits the safe height of the structure.

While it is true that dams depending upon a gravity section for safety may require such dimensions that the line of maximum pressure in the structure will fall inside the middle third, the reasons for such sections should be based as nearly as possible upon the factors which control the action of the stresses generated in the structure by the loads to which it is subjected.

If a proper batter be given to the up-stream face of a dam, the maximum pressure on the foundation will be some distance from the face, and the masonry subjected to such pressures situated so as to withstand greater stresses than that on the surface of the structure.

The distribution of pressure on the foundation of a dam, for variation in the cross-section of the structure, may be estimated from the following diagram:




Let a, b, c, h, g, f, e, d, be a section of a dam 1 ft. thick, resting on a solid-rock foundation, a-b, and having similar slopes on each face of the structure.

In this case, if the structure were absolutely rigid, the weight would be uniformly distributed over the base, a-b; but since the masonry and foundation will yield slightly under heavy loads, the maximum pressure will be at the center, m, gradually diminishing to the ends of the section at a and b. If, instead of being a symmetrical section, the portion b, c, h, i, k, be removed, the pressure on a-k will be changed only to the extent due to the redistribution of that portion of load supported on beam between b and k, the line of maximum pressure would still be a considerable distance from the heel of the dam, and the pressure at the toe would be much stronger than shown by the usual analysis.

If the section a, d, e, f, g, h, i, k, be cut by vertical planes spaced 1 ft. apart, and the columns thus formed be supposed free to move, each square foot of the foundation would support a weight equal to that of a column of masonry of the same height as that of the dam vertically above the point considered, the maximum pressure being directly below f-g.

In the construction of a concrete or masonry dam, the material as deposited in the structure is sufficiently pliable that for a depth of 20 to 30 ft. above the foundation, the pressure on all parts of the base will practically vary with the depth of material, but as the mortar sets and hardens, this foundation course acts as a beam resting on material which will yield only slightly under heavy loads, and partly distributes the pressure arising from the weight of the upper portions of the dam throughout the foundation.

The maximum pressure on any part of the foundation will vary with the elasticity and compressibility of the masonry and foundation, but under average conditions it should not exceed three-fourths of the weight of a column of masonry of the same height as the dam.

In short dams having ends firmly anchored in the side walls the structure acts as a horizontal beam, and the pressures developed on the base are far different from those determined by the usual method of considering the water pressure and the weight of the structure to act as a couple.

In high dams, where the length near the base does not exceed 200 ft. the width is so great that, with the ends firmly anchored in the side walls, the lower portion of the dam acts to a great degree as a horizontal beam of such magnitude that very little deformation is possible, and consequently only a small portion of the stresses from the water pressure are transmitted to the foundation. The upper portion of the structure being longer and of less width, distortion will take place, and the stresses arising from such movement will be distributed through the base of the dam to the foundation, and to the side walls, according to the design of the dam.

The strains arising from changes of temperature in straight dams are sometimes enormous, and unless provided for with large coefficients of safety, may endanger the structure. The base of a dam being massive, and generally submerged in water, has a much less range of temperature than the upper portions of the structure, which in many cases may exceed 100° F.

The expansion of sandstone for 100° change of temperature is about 1 in. in 90 ft., limestone about 1 in. in 125 ft., and granite about 1 in. in 170 ft. The expansion of masonry and concrete will vary with the material of which composed, but will probably average about 1 in. in 125 ft. for 100° change of temperature.

A dam 500 ft. long in passing through a change of 100° temperature, will expand or contract about 4 ins. in its entire length if not prevented by the side walls, and subject the upper portion of the wall to greater strains than those due to water pressure and weight of the dam, which the structure was designed to withstand. Internal stresses in the masonry from this cause probably have had much to do with some of the dams that have failed. A drop of 20° in the temperature at the crest of the dam, may cause tensile strain of 40 tons per square foot, and, unless supported by arch construction, leave the dam with only the resistance to shearing and sliding to withstand the water pressure.

In the problem of constructing a dam to safely withstand the pressure of a given head of water against its face, the structure can either be made sufficiently massive to resist shearing, overturning, and sliding from the water pressure, or if of not too long a span, may be built as an arch and the stresses developed by the water pressure resisted by the side walls against which the ends of the arch abut.

It has been stated by high authorities that the masonry of a dam is so rigid and non-elastic that it cannot act as an arch "for the reason that if the profile is sufficient to stand the water pressure by its own gravity, the mass will not yield under pressure, and if it does not yield, no pressure can be transmitted to the sides through the curve."

If this assumption is correct, is it not equally true that if the mass does not yield under the applied pressure, no pressure would be transmitted in a vertical plane to the toe of the dam? The transmission of pressure to the base of the dam from horizontal water pressures on the face, presupposes deformation of the structure, and if the crest be supported by an arch abutting against the side walls, a portion of the pressure will be taken up by the resistance of the arch.

Since the same principle of the transmission of force is involved, whether the pressure is transmitted through the arch to the side walls, or in a vertical plane to the toe of the dam, it is safe to assume that under normal conditions in an arch dam, a portion of the pressure will be transmitted to the side walls, and the balance to the toe of the dam, the amount of which will vary with the tensile or compressive strains in the arch arising from changes of temperature.

If the upper portion of the dam be constructed during a period of warm weather, the arch will be under tension during the cooler portions of the year, and a large part of the pressure from the water will be transmitted to

*A paper prepared for the 11th National Irrigation Congress, held at Ogden, Utah, September, 1903.

†Consulting Engineer, 39 West Canfield Ave., Detroit, Mich.

the toe of the dam, whereas if the walls be constructed in cool weather, the arch will be under compression most of the time, and the water pressure will be transmitted directly to the side walls by the arch.

It is therefore desirable that the upper portion of arch dams should be constructed during the season of the year when the temperature will average below the mean for the locality of the structure. Assuming that the nature of the masonry in a dam will permit it to act as an arch to only a limited extent, the structure becomes practically an arch beam supported at each end, and without regard to weight, will be safe as long as the shearing stress produced by the horizontal pressure of the water is not sufficient to produce rupture of the dam section at the side walls.

In most cases for arch dams of less than 500 ft. length on the crest, the cross-section of the structure necessary to keep the pressure on the foundation within safe limits, is ample to withstand the shearing stress from the water pressure, making the arch type of dam, for the same stability, much more economical than one of gravity section.

In some experiments made during the construction of the St. Louis bridge, it was found that the magnesian limestone used in the piers had practically the same modulus of elasticity as that of wrought iron, viz: 15,000,000 to 20,000,000, and although no determination has yet been made of the elasticity of the sandstone at the site of the Salt River dam, it is probable that the modulus is not much different from that found for magnesian limestone at St. Louis.

In a dam of 25 ft. or less thickness at the top, and having elasticity equal to that of ordinary rocks or concrete, a large percentage of the pressure of the water against the face of the dam will be transmitted directly to the side walls by the arch, and any such dam with sufficient cross-section to stand the shearing stress from water pressure on the face, will be absolutely safe, without taking into consideration the weight of the dam on the foundation. In other words, if such a dam was supposed to be supported entirely by water pressure from under the foundation, the structure would still be secure.

The best construction for an arch dam is probably a wall having faces of first-class masonry with center filling of concrete and large rock. An arch dam of such construction will consist practically of two concentric arches capable of withstanding heavy pressures, connected by a water-tight concrete core, or web, having a much lower modulus of elasticity—a condition well adapted to meet all of the requirements of the problem, and at the same time eliminate the necessity of molds and false works in placing the concrete in the structure. In cases of high dams of considerable length the use of railroad iron in the upper portion of the structure may be necessary to prevent temperature cracks.

The internal stresses in a large body of masonry arising from inequalities of its elasticity in its different parts, from unequal distribution of its own weight on its foundation, from compressive and tensile forces developed by changes of temperature in different parts of the structure, and from resistance to external pressures from wind, water and shocks, cannot be absolutely determined; but with a correct knowledge of the weight, strength, elasticity and expansion of the materials with which constructed, the movement of the crest of the dam, for changes of temperature and water pressure, can be stated sufficiently near for determining the thickness of the dam and the radius of the horizontal section best adapted for distributing the stresses between the base of the dam and the side walls against which the structure abuts. In any arch dam the thrust in any horizontal section is equal to the pressure on a unit of surface multiplied by the length of the radius, and the compression which takes place in the arch is equal to twice the deflection into the tangent of one-fourth of the arc covered by the section considered.

The mid-channel section of the dam may be regarded as a vertical cantilever beam supported rigidly at its base, and composed of material of which the weight and modulus of elasticity are known.

The application of pressure to the up stream edge of the section will cause a deformation of the structure and a slight movement of the crest down stream, which from the known nature of materials and design of construction, can be computed within reasonable limits.

This movement of the crest corresponds with the deflection in the horizontal arch, and the section instead of being a vertical cantilever beam becomes a beam rigidly fixed at the base with its free end supported by an arch having a slight elasticity, and if the radius of the arch be properly proportioned with regard to the height of the dam, the length of the crest, and the modulus of elasticity of the materials of construction, the stresses from the water pressure will be transmitted to both the base of the dam and the side walls,practically giving the structure more than twice the stability of a gravity section dam of the same dimensions. The height at which a dam may safely be built at any given location, does not depend so much on the water pressure to which it will be subjected, as upon the nature of the material of which constructed and the changes of temperature which will occur in the masonry. In a dam 250 ft. high and 500 ft. long on the crest, the tensile and compressive strains due to the ordinary range of temperature may be four times, or more, those arising from the water pressure with the reservoir full, and, in a straight dam, are liable to open the joints of the masonry, and in an arch dam, will cause compressive stresses in the masonry at the base and side walls as great as is safe to place on ordinary materials of construction.

In the analysis of a recent structure, it is stated that precedents are of greater importance than theoretical analysis of the stresses in determining the safety of the dam. The fact that a dam has not failed is by no means a safe indication that it has been correctly designed, and to follow such methods of construction, without original investigation of all the conditions involved in each case, is certain to bring the engineer to grief.

While it is true that the earlier investigators in their endeavors to determine the laws governing the action of stresses in dams were obliged to make certain assumptions, the facilities afforded the engineer of the present time for determining the strength and physical characteristics of all materials of construction largely eleminates such necessity. In case any problem should arise where certain general assumptions are necessary, it may be stated as a general principle that the nearer such assumptions are made to the true conditions and facts, the more nearly will the results obtained conform to those which would result from an absolutely accurate determination of all the factors involved.

## THE PROPOSED UNDERGROUND MOVING PLATFORM ACROSS THE WILLIAMSBURG BRIDGE TO BATTERY PARK, NEW YORK CITY.

Among the numerous plans proposed for transporting passengers across the three East River bridges now completed and under construction at New York City the most novel is the plan to run moving platforms across the bridges and through subways in Manhattan to suitable connections with the present subway and elevated railways. Until recently this plan, although favorably mentioned by a number of prominent engineers, seemed to present no great likelihood of being seriously considered. Under the present laws all plans for transportation which involve underground construction in New York City must be reviewed and approved by the Board of Rapid Transit Railroad Commissioners. When, therefore, a proposition was made some months ago to build a moving platform across the Williamsburg bridge and thence underground in Manhattan to Bowling Green it came before the Commissioners for review and was referred by them to their Chief Engineer, Mr. Wm. Barclay Parsons, M. Am. Soc. C. E. Recently Mr. Parsons made a report to the Commissioners on the proposition, and as a result of this report the Extensions Committee of the Commission voted on Sept. 24, 1903, to recommend to the whole Board the adoption of a plan to build a subway for moving platforms from the Williamsburg bridge terminal in Manhattan to Battery Park. Before the actual construction is authorized the committee's plans will have to be approved by the Commission, then be passed by the Board of Aldermen and the Board of Estimate and Apportionment, and finally be approved by the Appellate Division of the Supreme Court. This procedure having been carried out, bids open to all will have to be asked for the construction of the subway.

Sketch Map Showing Route of Proposed Underground Moving Platform for Passenger Transportation Between the Williamsburg Bridge and Battery Park, New York City.

The full details of the proposed construction are not yet available, but some of the general features have been announced and can be given. The route of the proposed subway is shown by the accompanying sketch map. As will be seen, it continues the bridge line along Delancey St to the Bowery, and then follows the Bowery, Park Row and Nassau St. south to Broad St., whence is bends east to South St., which is followed to the Battery. As will be seen from the map, the route is close to the main north and south routes of travel in lower Manhattan and passes close to the terminals of the new Manhattan and the old Brooklyn bridges, so that when the time comes they may readily be coupled with the system operating the Williamsburg bridge. At the Battery connection will be made with the Battery loop of the Rapid Transit Railway extension to Brooklyn.

According to the published accounts the subway will be constructed much like the regular Rapid Transit Railway work, so that in the event of the failure of the moving platform plan to work satisfactorily it can be replaced with standard double track and electrically operated trains like the remainder of the Rapid Transit line. For the moving platform the subway will be divided longitudinally by an axial partition, on one side of which will be the uptown platform and on the other side the downtown platform. The platforms will probably have three stages, and the last will be provided with cross-seats seating three persons each and spaced 3 ft. apart. Electricity will be the motive power, and all construction will be made fire-resisting so far as is practicable.

## ELECTRIC MOTORS FOR CENTRIFUGAL PUMPS AND FANS.*

By A. J. Bowie, Jr.,† Assoc. Am. Inst. Elec. E.

The problem of determining the proper capacity of a motor for driving centrifugal pumps and fans, is not always easy to solve. The solution is too often the result of guess-work, and is liable to be seriously in error. In the case of direct-acting pumps working against a known head, the proper motor capacity can be easily calculated. The most economical size of motor for the purchaser, however, may not be solely determined by the possible loads, and their duration, which the motor may be called upon to carry. Particularly is this true where a flat rate system is in vogue and where power is purchased at a price depending upon the rating of the motor and not upon the power actually used. This system of charging for power leads to great abuse in the underrating of motors; on the other hand, it may work a great hardship on a purchaser of power who innocently uses a motor far larger than is necessary and pays for power that is not consumed. Several instances of the latter have come under the writer's observation, and the system in his opinion should be condemned. Where measurements of power are taken as a basis for determining the flat rate, such measurements should be taken under the average running conditions. A variation in these conditions may lead to a very erroneous system of charges in the case of centrifugal pumps and fans as will be seen later.

For centrifugal pumps and fans the power required for operation at a rated speed and head is by no means all that need be known. One should also know the maximum load which can ever come on the motor when driving the pump or fan at the rated speed and under any possible head. This maximum may be far in excess of the rating.

The problem in case of centrifugal pumps and fans will be treated under the following three heads: (1) Constant speed and varying heads; (2) changing speed with head varying directly as the square of the speed; and, (3) constant head and varying speed.

1. CONSTANT SPEED.—The following treatment ap-

*Slightly condensed from a paper presented before the American Institute of Electrical Engineers, Sept. 25, 1903.

†Engineer, Electrical Department, Union Iron Works, San Francisco, Cal.

plies equally well to centrifugal fans, as the same laws govern the flow of water and air, but pumps alone will be considered. Represent by H the head generated by the pumps; this is equal to the difference between the discharge and suction pressures, plus the difference between the discharge and suction velocity heads. Represent by Q the cubic feet of water discharged per minute. The general equation expressing the relation between Q and H is,

$$H = A - B Q^2 + C Q,$$

where A, B, and C are constants. This is the equation of a parabola, and it will approximately represent the actual curves obtained in practice. Back-slippage though the running wings of the pumps has not been taken into account in the above equation, and will account in part for discrepancies between theoretical and actual results.

The pressure exerted by the pump is a maximum when

$$\frac{d H}{d Q} = 0,$$

hence, when

$$2 B Q = C, \text{ or } Q = \frac{C}{2 B}.$$

Therefore

$$H = A + \frac{C^2}{4 B},$$

is the maximum value. Let W = output of pump in watts. W = K H Q; K being a constant.

$$W = K Q (A - B Q^2 + C Q).$$

This is a maximum when

$$\frac{d W}{d Q} = 0,$$

when $A - 3 B Q^2 + 2 C Q = 0$. Then the maximum

$$Q = \frac{C}{3B} \pm \sqrt{\frac{A}{3B} + \frac{C^2}{9B^2}},$$

corresponding to $H = \frac{2}{3} A + \frac{1}{3} C Q$.

In some pumps the constant C is so small that it may be disregarded. In this event the maximum head is obtained at zero discharge and is equal to A. The maximum output of the pump is obtained at a head of $\frac{2}{3}$ A. The relative values of C and B determine the nature of the output curve of the pump and the maximum obtainable pressure. If B be fixed, the greater the value of C the greater will be the capacity of the pump. If C be fixed, the less the value of B the greater will be the pump capacity. However, the relative values of C and B will have a very important bearing on the ratio between the maximum power required to run the pump, and the power required to run it at the point of highest efficiency, or at the point of maximum output.

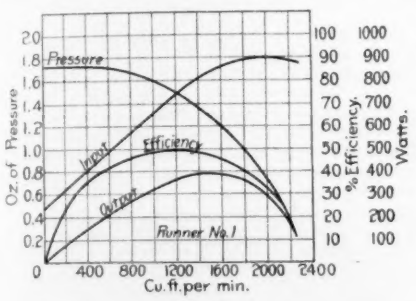



Fig. 1.

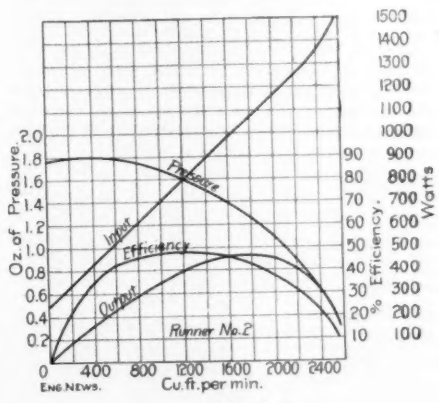



Fig. 2.

FIGS. 1 AND 2. CHARACTERISTIC CURVES OF CENTRIFUGAL PUMPS; DIFFERENT RUNNERS IN SAME CASE.

The quantity $\frac{C}{B}$ is influenced largely by the angle which the vanes of the pump make with the radius, at the external periphery, as well as by the width of the face of the runner. However, it is beyond the scope of this paper to enter into a discussion of the design of pumps, other than in a general way so as to show the effect of the design upon the maximum power required to drive the pump. Usually this power will increase as the effective head of the pump decreases. This would naturally be expected as long as the energy output of the pump is on the increase, but after the latter starts to decrease it might be considered natural to look for a decrease of the driving power. However, when B is fixed, the smaller the value of C the less will be the tendency of the pump to require additional power to drive it at heads lower than its rated capacity, and the higher the value of C, the greater will this additional power become. When C is fixed, the reverse holds true for B.

A centrifugal pump, when operated at too low a head, shows a very low efficiency. In general, however, with a well-designed pump, the range of efficient operation for a constant speed is fairly broad. The principle point which the writer wishes to bring out in this connection is the danger of overloading a motor through too low a head. A manufacturer will usually guarantee a certain efficiency and input for a pump at its rated load, but will say nothing concerning the power required to drive it at heads lower than that called for. The question may arise, why not give a pump the proper speed for a given head. This is frequently impracticable, since pumps must often be installed to work under widely varying heads, where it is not an easy matter to change the speeds. Oftentimes the heads at which the water must be elevated may vary practically from zero to the maximum head against which the pump will deliver. In some cases these overloads may be of long duration, and a sufficiently large motor should be installed. On the other hand, the maximum load will often be only of short duration, as for example in the case of pumping from a well, particularly where there is a pit in connection with it, which is partially filled up with water. Generally it will not take the pump long very decidedly to lower the water, and the duration of overload will be brief.

The behavior of centrifugal pumps may be represented very completely by four curves. These curves are based upon an absolutely fixed speed. Corrections for variations from this speed may be made without difficulty. These curves have, in each case, for abscissæ, the volumes of the fluids delivered in a fixed time interval, while for ordinates they have respectively, head, watts input to pump, watts output from pump, and pump efficiency. The last two curves determine the proper rating of the pump. In general, the maximum efficiency occurs before arriving at the point of maximum output. The proper rating is somewhat indefinite, as will appear from the appended curves obtained in practice. The natural tendency of a manufacturer is to rate his pumps as high as possible. Still, he does not care to go far beyond the point of maximum efficiency at the full load rating.

The accompanying curves (Figs. 1 and 2) show the results of tests on two fans with the same casing, but with different runners. In the first set $\frac{C}{B}$ is smaller than in the second set. According to the curves, the first fan is of smaller capacity, but of higher efficiency than the second. Particularly noticeable is the curve of watts input, which in the first case shows a decided concavity towards the X axis, shortly after passing the maximum output of the pump, while in the second case the corresponding curve becomes slightly convexed towards the X axis, and requires far more power under low heads. Thus the ratio of maximum outputs is 1.15, and the ratio of maximum inputs, within the range of the tests, is 1.68. These fans represent by no means extreme conditions. In the case of the second fan, if it should be called upon to operate under the lowest head tested, and if it were rated at the point of maximum efficiency and a corresponding motor were installed, then this motor would be liable to an overload of 89%.

2. HEAD VARYING AS THE SQUARE OF THE SPEED.—If a centrifugal pump delivers Q cubic feet of water per minute, against a pressure of H feet while running at R revolutions per minute, then if the speed be altered to $R_1 = K R$, it will deliver $Q_1 = K Q$ cubic feet per minute against a pressure $H_1 = K^2 H$ feet. If W = watts output of pump in the first case, and I = watts input to pump, then $W_1 = K^2 W$, and $I_1 = K^3 I$. These equations follow from the proportionality between the head generated and the square of the speed, and because, when the volumetric output varies directly as the speed, the relative angles of motion of the water are not altered. Furthermore, all frictional losses vary as the cube of the speed, and all volumetric leakage losses vary as the speed. Hence it follows that increasing the speed of the runner, where the effective head varies as the square of the speed, will result in increasing the capacity of the pump directly as the speed, and increasing the power input and output directly as the cube of the speed. The conditions presupposed in this case, namely that the head varies as the square of the speed, is the condition which exists in the case of ventilating fans, where the head is used up in friction in the pipe, and in the velocity head of discharge from the pipe. This velocity head, and also the friction head vary as the square of the discharge from the pipe.

What concerns the motor capacity particularly is the fact that the power varies directly as the cube of the speed. Hence the $I^2 R$ loss in the armature varies as the sixth power of the speed.

3. CONSTANT HEAD.—This is the condition which exists when pumping against a fixed lift, when the velocity heads lost at entrance and discharge and the frictional losses in the pipe may be disregarded in comparison with the lift.

In this case, and likewise in case 2, the characteristic curves of a pump under variation of speed can be obtained from those at another speed by multiplying the quantities discharged by the ratio, K, of the speeds, the pressures by $K^2$, and the watts input and output by $K^3$. The efficiencies remain the same. (If the head remains constant, as assumed for case 3, the preceding statement is evidently not true for pressures and watts, and the conclusions of the following paragraph are also invalidated.—Ed.)

It is evident that the apparatus of case 3 will operate at a point situated further along on the curves than that of case 2; the result being a greater delivery of water, and, usually, a greater input to the pump. In general this input will vary faster than the cube, and not as fast as the fourth power of the speed. As an approximation say as the $3\frac{1}{2}$ power of the speed. This makes the $I^2 R$ loss in the armature vary as the seventh power of the speed. Thus 10% increase in speed represents 40% increase in power and 96% increase in the $I^2 R$ loss in the armature.

From this it may be seen what an extremely large effect speed variation has on motors driving centrifugal apparatus. There is perhaps no other form of apparatus so sensitive to change of speed in its power consumption. The effect which a steady pump load has in holding the voltage constant by preventing the racing of generators is thus explained. If centrifugal apparatus be provided with induction or synchronous motors then the speed is practically fixed by that of the prime mover so that the maximum load for the motor with a given pump or fan is determined. Speed control of direct-current motors by variation of field resistance has a limited range in practice and has many points which tend to make it objectionable for the best conditions of commutation. Even at best there will be considerable field distortion at the high speeds. The effect of a load which would increase as the cube of the speed, and even faster would seem to limit still further the allowable speed variation to be obtained in this manner. In view of these facts it is well to provide a motor of ample size for such work and at the same time to see that the field rheostat cannot cut in so much resistance as to overload the motor. As has been shown, the volumetric output of a centrifugal pump varies directly as the speed of revolution. In spite of this fact, many manufacturers rate their pumps at a fixed volumetric output, and publish a list of suitable speeds for different heads under which it may be desired to operate the pump. In some cases the highest speed given in these lists is three or more times as great as the lowest speed. Such misinformation leads to a great deal of trouble and cannot be too strongly condemned.

In regard to the statement previously made, that the efficiencies of pumps and fans were independent of the speed, proper changes being made for head and discharge, no consideration was taken to the effect of unbalance of the runner or of bearing or stuffing-box friction. These, of course, would modify the results obtained. Lack of proper running balance is very apt to cause serious loss of power.

## REPAIRS OF SUBMARINE WATER PIPES.*

By Thos. R. Cook.†

My experience in this work while not extensive, has been quite varied. I have observed that the contemplation of a perplexing or apparently difficult job, is more harrowing to one's nerves than the actual doing of the work.

We have in our city two 30 and two 16-in. force mains under water. The two former but a short distance, one of the latter for about 1,600 ft., all from 18 to 30 ft. below average stage of water.

When you have but one force main supplying any considerable portion of your city, it is imperative, of course, that you have it shut off as short a time as possible and often it is possible that you can make temporary repairs until such time as you can get your patterns and castings out to make a permanent job. The first difficulty of any moment you will encounter is removing the earth from on top and around your pipe so that you can get at it, and it generally saves time in the end to do this job thoroughly and well, so that you will have plenty of room. I find the best method for doing this work is to employ a so-called "sand-sucker," if one can be procured, which in our locality we have no difficulty in doing, which

*Abstract of a paper read before the Central States Water-Works Association, September, 1903.

†Superintendent Water-Works, Toledo, O.

with their 10-in. suction pipe and large centrifugal pumps do the required work very quickly.

Small leaks in joints can often be stopped by driving in copper wedges; also tea lead that can be calked cold. Where the pipe is worn through by sand, a half sleeve can be applied.

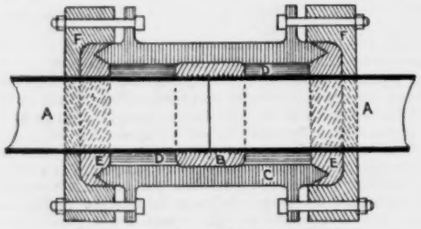
The diver should use great care in calipering the pipe at both ends of where a sleeve will be placed. If there are lumps or rough spots on these ends they should be chipped off with a cold chisel. The flanged half sleeves should have holes drilled about ¾-in. deep and 3½ ins. from the end of sleeve, to hold the lead in when the sleeve is poured. When the sleeve is on the planer a groove should be cut in the lower flange—say ¾-in. at the top and tapered to ¼-in. at the bottom of the groove. The top flange should be fitted with a corresponding boss so that it will fit into the groove. This should be between the bolt holes and the inside of the flange. This not only secures a better joint, but it enables the diver to place the castings in their proper positions more readily.

When the sleeve is prepared select a piece of pipe that is just the diameter of your submerged pipe, clamp the sleeve on and pour the joints, being careful to put shims between the castings to keep them apart the thickness of the lead gaskets, which is known as 6 or 8-lb. lead. Then take a key-hole saw and saw your lead apart between the flanges, the shims that you have previously put in giving you plenty of room for this purpose. Then take your casting off and you have it all ready for the diver to put on your submerged pipe. In large-sized pipe an eye-bolt should be tapped in at both ends of the top casting, to make fast to when the casting is lowered into position. After the bottom casting is lowered and placed under the pipe, it should be blocked up to the pipe firmly, the top casting lowered, bolts put in and screwed up firmly and then all that remains to be done is to calk the lead in the ends of sleeve.

I recently applied a sleeve to a 16-in. pipe that covered a ball joint in the pipe. The sleeve was 42 ins. long and weighed something over a ton.

## PIPE JOINT FOR A SUBMARINE MAIN.

At the annual meeting of the Western Gas Association, Mr. J. A. Mayers, of New York, described a pipe joint used by him in laying a 12 in. main across a river, and which proved tight under 200 lbs. pressure. This device is shown in the accompanying cut. It is essentially a cast-iron screw sleeve coupling, with a rubber gasket at each end. In the cut, AA, are the two lengths of pipe, B is a wrought iron threaded sleeve coupling cast in the cast-iron sleeve C. The space D D between the sleeve and the pipe is filled with lead, and the rubber gaskets E E are drawn into place by bolting up the end rings F F, thus compressing the gaskets against the sleeve, the lead and the pipe.




Pipe Joint for Submarine Main.
J. A. Mayers, New York, Designer.

A LOW-GRADE CUT-OFF RAILWAY CONNECTION is being built by the Atchison, Topeka & Santa Fe Ry. to facilitate trade on its line to the Pacific coast. The present line runs west from Kansas City across Kansas and as far as La Junta, Colo., then turning south to Albuquerque, N. Mex., where one main line runs west to the Pacific and another goes south to El Paso, to connect with the Mexican Central Ry. A diagonal line runs southwest across Kansas, Oklahoma and Texas, extending south to Pecos City. The section of line between La Junta and Albuquerque is difficult and expensive to operate, as it reaches elevations of 7,607 ft. and 7,421 ft. at Raton and Glorieta, with grades of 184.8 ft. and 158.4 ft. per mile, respectively. It has now been decided to build a line east from the Pacific coast line, starting 30 miles south of Albuquerque and extending eastward through the Abo pass to a point near Texico, on the Kansas City & Pecos line. This cut-off will be about 250 miles long, with maximum grades of 31.68 ft. per mile, except for about 25 miles near the summit, where the grade will be 66 ft. per mile.

THE ELIMINATION OF GRADE CROSSINGS has been one of the features of the work of reducing grades and making other improvements on the Norfolk & Western Ry. The annual report states that numerous grade crossings of highways have been abolished, and that arrangements have been made with electric railways by which several grade crossings were avoided.

LARGE COAL CARS are gradually coming into use on English railways in spite of frequent statements to the effect that the old cars of 8 and 10 tons capacity are best suited to British railway conditions. Several railways are using 15 and 20-ton cars and a few are using 40-ton cars, especially for hauling coal direct from coal mines to shipping ports. The Northeastern Ry. uses 40-ton cars for this class of service and 20-ton cars for general use. The Great Eastern Ry. has also built at its shops 100 coal cars of 20 tons capacity, of steel construction throughout. These were designed by Mr. James Holden, Locomotive Superintendent. They are 21 ft. 6 ins. long, 8 ft. 2 ins. wide and 4 ft. 7 ins. deep, with a cubic capacity of 800 cu. ft. The car is mounted on four 36-in. wheels, with 12-ft. wheelbase; and the journals are 5 × 13 ins., with a plate spring over each axle box. The side and end plates are 3-16-in. thick, and the floor plates ¼-in. thick. The net weight is 8¼ tons as compared with 6¼ tons for the cars of 10 tons capacity; and a train of 20 cars of 20 tons capacity is only 493 ft. 4 ins. long, as compared with 726 ft. 8 ins. for a train of 40 cars of 10 tons capacity, both trains carrying the same weight of coal. The dead load is 29% and the paying load 71% of the gross weight. The Furness Ry. is building 20-ton hopper cars for the ore traffic, to replace old 5-ton cars.

A FOUR-CYLINDER BALANCED COMPOUND LOCOmotive is now being tried on the Atchison, Topeka & Santa Fe Ry. It is a passenger engine of the Atlantic type, with wide firebox over the frames. The high-pressure cylinders are under the smokebox, and drive a 10-in. crank axle with 10-in. crank pins, the journals being 8⅝ × 10 ins. The crank cheeks are 4 ins. apart, 5 ins. thick and approximately rectangular, 20 × 25 ins. The low-pressure cylinders are outside, and drive the wheels on the same axle. The engine was built by the Baldwin Locomotive Works, and its general dimensions are as follows:

| | | |
|---|---|---|
| Wheels, driving | 6 ft. | 1 in. |
| Wheelbase, driving | 6 " | 4 ins. |
| Wheelbase, total | 29 " | 6 " |
| Weight on driving wheels | 101,400 | lbs. |
| Weight, total, engine | 193,760 | " |
| Weight, engine and tender | 348,000 | " |
| Cylinders, h.-p. (2) | 15 × 26 | ins. |
| Cylinders, l.-p. (2) | 25 × 26 | " |
| Boiler, diameter | 5 ft. 6 | " |
| Firebox | 9 ft. × 5 ft. 6 | " |
| Heating surface, tubes | 2,893 | sq. ft. |
| " " firebox | 190 | " |
| " " total | 3,083 | " |
| Grate area | 49.5 | " |

THE MOST SERIOUS RAILWAY ACCIDENT of the week was the derailment of a fast mail train on the Southern Ry. at Danville, Va., on Sept. 27. The train left the rails while running around a sharp curve at high speed, just at the beginning of a high trestle over a creek; the entire train was destroyed by its fall into the valley, and nine of the sixteen persons on board were killed outright. The first report was that the accident occurred on the trestle; a statement issued by Mr. W. W. Finley, Vice-President of the Southern Ry., corrects this and states the facts as known up to the present:

The train jumped the track about 40 ft. north of the trestle on sound track, in good line and surface and proper gage. The trestle was in first-class condition and was so well braced that the engine and train falling on the outside of the curve knocked down the outside posts and left the others standing. Eye-witnesses report that the train was approaching the trestle at a rate of speed of 30 to 35 miles an hour. The cause of the accident is being investigated.

The trestle was repaired and was again in service on the morning following the accident. Another unusual accident occurred on the same railway a few days before. Two freight trains collided at a crossing at Boone, near Portsmouth, Va., on Sept. 21, killing one man. It appears that the crossing was provided with interlocked signals, but it is claimed that the signals were thrown while the first train was on the crossing.——A crossing accident, in which an electric car figures, is reported from Marion, Ind. An interurban car ran into a train on the Cleveland, Cincinnati, Chicago & St. Louis Ry. on Sept. 24; several persons were killed in the wreck.

THE TURBINE STEAMER "BRIGHTON," built for the Newhaven and Dieppe channel service of the Western Ry., of France, and the London, Brighton & South Coast Ry., of England, has just been put in service. It is very similar to the "Arundel," which has been in service for some months, but certain improvements have been introduced. It is 282 ft. long and 34 ft. beam. There are three sets of Parsons steam turbines, steam at 160 lbs. pressure being furnished by Scotch marine boilers. Under ordinary conditions the shaft of the one high-pressure turbine will make 520 revolutions per minute, and the shafts of the two low-pressure turbines will make 600 revolutions. The former has a screw of 5 ft. 6 ins. diameter and 6 ft. pitch; the latter has screws of 5 ft. 7½ ins. diameter and slightly less pitch. With these speeds the vessel makes about 21½ knots. In handling the steamer at the docks, etc., the central shaft is not driven and the two side shafts are used like twin screws to assist the steering. The vessel was built by Denny & Bros., of Dumbarton, Scotland.

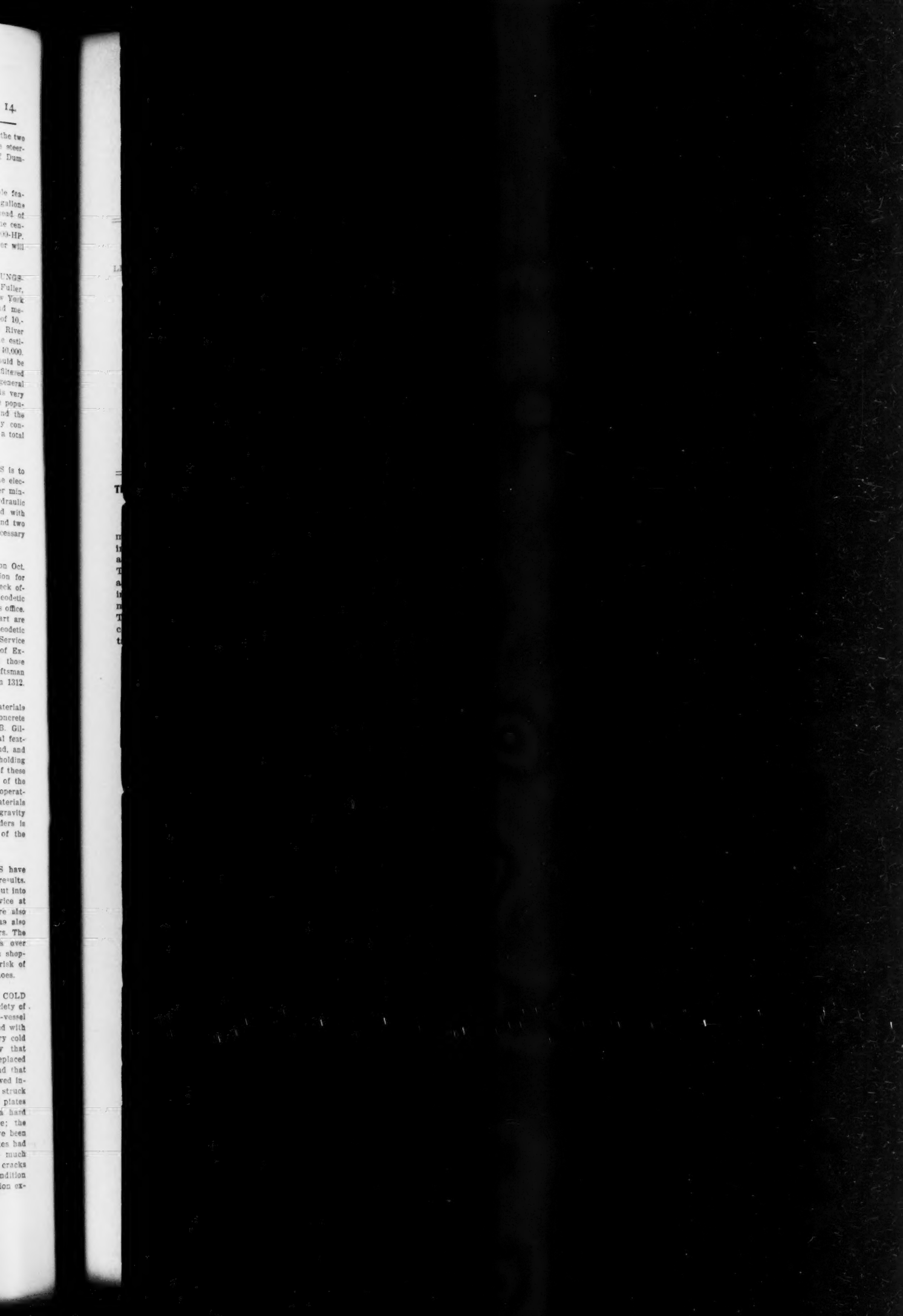
A TRIPLE CASCADE will be one of the notable features of the St. Louis Exposition. About 90,000 gallons of water per minute will be pumped under a head of 159 ft. by means of three 36-in. single-stage turbine centrifugal Worthington pumps, each driven by a 2,000-HP. Westinghouse alternating-current motor. The water will be taken from and discharge into the grand basin.

A WATER PURIFICATION PLANT FOR YOUNGStown, O., has been recommended by Mr. Geo. W. Fuller, Assoc. M. Am. Soc. C. E., of Hering & Fuller, New York City. Small settling and clear water basins, and mechanical filters, the latter with a daily capacity of 10,000,000 gallons, are proposed, and the Mahoning River would be continued as the source of supply. The estimated cost of the works, as just outlined, is $140,000. Later on, more storage capacity for clear water would be required, and also large storage reservoirs for unfiltered water. The city, Mr. Fuller states, has a low general death rate, but the mortality from typhoid fever is very high. It is estimated that not more than half the population of Youngstown now receives city water, and the records show that the present total average daily consumption is a little less than 3,500,000 gallons for a total population of about 60,000.

WATER POWER FOR ALASKAN GOLD MINES is to be developed in the Saw Tooth Mountains, and the electricity generated will be carried to Nome and other mining districts of Seward Peninsula to operate the hydraulic placer plants there. Water in great volume and with falls of 700 to 1,800 ft. is said to be available, and two engineering parties are in the field making the necessary surveys.

CIVIL SERVICE EXAMINATIONS will be held on Oct. 21 by the United States Civil Service Commission for eligibles to fill the following positions: Aid and deck officer, U. S. Geodetic Survey; computer, U. S. Geodetic Survey; engineer draftsman, Supervising Architect's office. The salaries attached to these positions at the start are $750, $1,000 and $1,400. Those wishing to take the Geodetic Survey examinations should apply to the Civil Service Commission, Washington, D. C., for the Manual of Examinations and blank form of application, and those wishing to take the examination for engineer draftsman should apply to the same body for application form 1312.

A DEVICE FOR MEASURING CONCRETE materials and for feeding them from hoppers into the concrete mixer has been put upon the market by Frank B. Gilbreth, 176 Federal St., Boston, Mass. The essential feature of the device is a drum that is revolved by hand, and by its revolution carries away from the hoppers holding cement, sand and stone a definite amount of each of these materials. This drum virtually forms the bottom of the chutes leading from each of the hoppers. One man operating this drum is said to be able to feed the materials faster than any existing plant can handle them. A gravity concrete mixer fed by one of these Gilbreth feeders is now in successful operation on the new works of the Columbian Cordage Co., at Auburn, N. Y.

BALL-BEARINGS FOR STREET-CAR MOTORS have been tried in Berlin, Germany, with satisfactory results. The first motors equipped with ball-bearings were put into use in 1901. Some 100 such motors are in service at present, generally on cars whose axle-bearings are also of the ball type. The Bremen street railway has also had some of its cars fitted with ball-bearing motors. The principal advantage of ball motor-shaft bearings over plain bearings is the saving in oil, the decrease in shop-time for relining bearings, and the reduction of risk of the armature coming into contact with the pole-shoes.

STEEL PLATES RENDERED BRITTLE BY COLD were described before the Hamburg (Germany) Society of Engineers by Mr. L. Benjamin. A steel lighter-vessel which had seen 10 or 12 years of service was towed with a fleet of barges through a thick field of ice in very cold weather. The lighter was battered so severely that it had to be docked, a number of broken plates replaced and others straightened. In this work it was found that many of the plates were extremely brittle and showed indications of strong internal stresses, so that when struck a sharp blow with the hammer several of the plates cracked. Test-pieces cut from the iron showed a hard material having a very coarse granular fracture; the material was so poor, in fact, that it could not hve been punched and riveted in that condition. As the plates had been in service for many years and had become much battered and dented during that time without any cracks appearing, it was thought that the dangerous condition of the metal was produced by the shocks and vibration experienced during extreme cold.